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COMBINED SEWER OVERFLOW SEMINAR PAPERS

Edison Water Quality Research Laboratory

November 1973

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This publication is a compilation of the papers presented at the seminar.

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COMBINED SEWER OVERFLOW SEMINAR PAPERS

A compilation of technical papers
and discussions presented at three
seminars in New York State given
jointly by the U. S. Environmental
Protection Agency and New York
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Conservation.

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FORWARD

The U.S. Environmental Protection Agency in conjunction with the New York State Department of Environmental Conservation conducted three one-day seminars on the problem of wet-weather flow pollution abatement. Many facets of the problem were considered including a brief overview of its magnitude and what the federal government is doing to manage and control this source of pollution. Various management, control and treatment techniques were described and the most up-to-date information on design and economics was presented. The audiience consisted of consulting and municipal engineers from all areas of New York State.

It is hoped that these seminars and this compilation of papers will help solve community problems or at least stimulate new ideas as to how storm and combined sewer overflow pollution abatement might be approached.

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SECTION I

STORMFLOW POLLUTION CONTROL IN THE U. S.

by

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I. PREFACE

In an effort to introduce this seminar and tie the various discussions you'll be hearing today together, I thought it would be appropriate to discuss the problem of stormwater discharges and combined sewer overflows from the Federal Government's involvement.

The nation-wide significance of pollution caused by storm generated discharges was first identified in a U.S. Public Health Service report published in 1964. Congress, in recognizing this problem, authorized funds under the FWPC Act of 1965 for the research, development and demonstration of techniques for controlling this source of pollution. Further authorization has been provided by the 1972 Amendments to the Act.

Hence, the Storm and Combined Sewer Overflow Pollution Control Program was originated and the problem of wet-weather flow pollution was classified into three categories:

1. Combined Sewer Overflows
2. Stormwater Discharges
3. Non-Sewered Runoff

To date over 116 grants and contracts totalling over $82,000,000 have been awarded , the Federal Government's share being in the neighborhood of $40,000,000 or 47.5%.

II. INTRODUCTION

The earliest sewers were built for the collection and disposal of stormwaters, and for convenience emptied into the nearest watercourse. In later years, house sewage was discharged into these large storm drains, automatically converting them into "combined" sewers. Subsequently, combined sewers came into widespread use in communities because they represented a lower investment than the construction of separate storm and sanitary sewers. (Fig. 1)

When the problem of pollution caused by sanitary or dry-weather discharges became recognized, the engineer was confronted with how to best separate the wet from the dry-weather flows to enable proper treatment of the sanitary sewage portion. This was overcome by designing overflow structures at selected points in the sewerage system, so that combined sewage flows greater than a predetermined multiple of mean dry-weather flow were discharged directly into the receiving stream. The diversion points were usually chosen close to the receiving water for economy, and new sewers were installed for intercepting and conveying the dry-weather flows to the sewage works for treatment.

These overflow or relief points may also be integral to separate sanitary systems. Initially, nominal allowances were made for infiltration and with pipe age this became more of a problem. Unauthorized connections compounded the problem, and reliefs in the "so called" separate sanitary system were used as an immediate and low cost solution. Studies conducted for the USEPA found that separate systems, with excessive infiltration and other inflows, act essentially as combined sewer systems.

III. COMBINED SEWER OVERFLOW PROBLEMS

The basic difficulty with combined and "nominal" sanitary sewers involves their "built-in" inefficiencies, i.e., their overflow points.

Untreated overflows from combined sewers, particularly during wet-

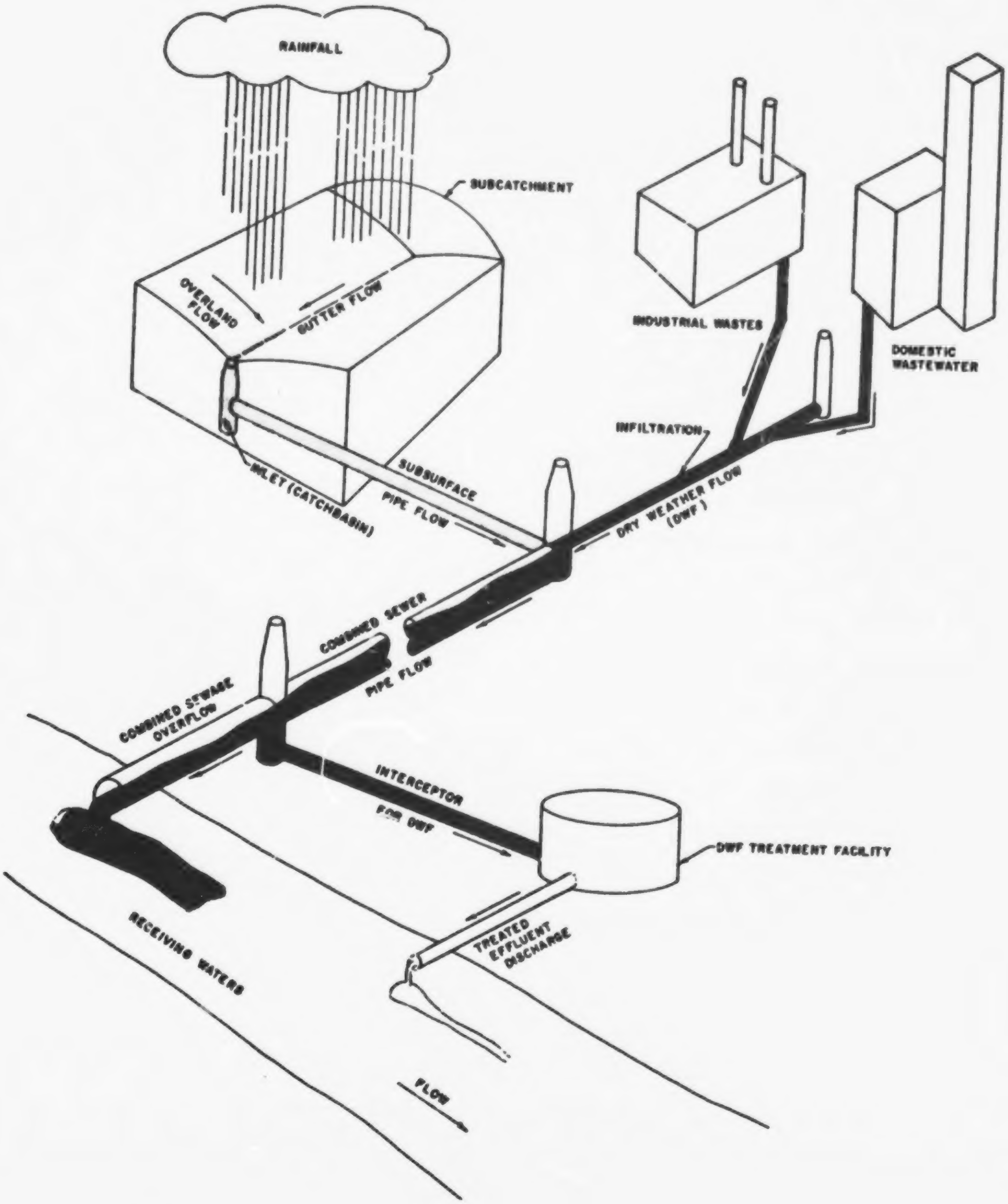

RAINFALL
SUBCATCHMENT
OVERLAND FLOW
GUTTER FLOW
INLET (CATCHBASIN)
SUBSURFACE
PIPE FLOW
INDUSTRIAL WASTES
DOMESTIC WASTEWATER
INFILTRATION
DRY WEATHER FLOW (DWF)
COMBINED SEWER
PIPE FLOW
COMBINED SEWAGE OVERFLOW
INTERCEPTOR
FOR DWF
DWF TREATMENT FACILITY
TREATED EFFLUENT DISCHARGE
RECEIVING WATERS
FLOW

Figure 1. SCHEMATIC SYSTEM DRAWING RAINFALL THROUGH OVERFLOW

weather, has proven to be a substantial pollution source in terms of impact upon receiving stream water quality--even though the percentage of sanitary sewage lost from the system by overflow is small, that is, in the order of 3 to 5 percent.

Pollution problems stemming from combined sewer overflows are widely distributed through the United States; the Northeast, Midwest, and Far-West being the principal areas of concentration. In a nation-wide survey performed by the APWA it was found that there are over 3,000,000 acres of combined sewer drainage area contained in more than 1,300 municipalities with a population of 54 million served by some 55,000 miles of combined sewers. Of 641 jurisdictions surveyed,

- 493 reported some 14,200 combined sewer overflow points,
- 340 reported infiltration problems during wet-weather and
- 96 indicated combined sewer overflows during dry-weather.

The magnitude of the overflow problem was exemplified by a 2-year study conducted on a 229 acre combined sewer watershed in Northhampton, England. This study showed that the cumulative yearly biochemical oxygen demand (BOD) load in the combined sewer overflows nearly equaled the BOD load contained in the effluent of the local secondary treatment plant. Suspended solids within the overflows were three times the load contributed by the treatment works effluent.

The relatively poor flow characteristics of combined sewers during dry-weather when sanitary wastes alone are carried, encourages settling and build-up of solids in the lines until a surge of flow caused by a rainstorm purges the system. Studies in Buffalo, New York have shown that 20 to 30 percent of the annual collection of domestic sewage solids are settled and eventually discharged during storms. As a result, a large residual sanitary pollution load, over and above that normally carried is discharged over a relatively short interval of time, oftentimes resulting in what is known as a "first flush" phenomenon. This can produce shock loadings detrimental to receiving water life.

Aside from the raw domestic and industrial sewage carried in the overflow, non-sanitary urban runoff in itself is a significant contributor to the overflow pollution load. As the storm runoff drains from urban land areas, it picks up accumulated debris, animal droppings, eroded soil, tire and vehicular exhaust residue, air pollution fallout, heavy metals, deicing compounds, pesticides and PCB's, fertilizers and other chemical additives, decayed vegetation, hazardous material spills, together with many other known and unknown pollutants. A study on a 1,067 acre drainage basin in Durham, North Carolina has shown that the annual BOD contribution attributable to surface wash from storms is approximately equal to that contribution of the secondary treated sanitary effluent and the total organic matter exhibited by chemical oxygen demand was estimated to exceed the amount in the raw sanitary sewage from a residential area of the same size.

It is important to note that there is no apt description of "typical" combined sewage or stormwater runoff characteristics due to the variable nature of the rainfall-runoff patterns. Figure 2 illustrates some general concentration ranges of the wastewater constituents listed. The major characteristic, i.e., qualitative variability, is shown by these data. Quality may range from superstrong sanitary sewage during the "first flush" to very diluted sewage later in the storm. The composition is dependent on a number of factors, including: length of antecedent dry weather, local climatic conditions, condition of the sewerage system and the nature of the drainage area.

FIGURE 2

CHARACTERISTICS OF COMBINED SEWER OVERFLOWS
(Selected Data)

BOD_5	-	30	TO	600	MG/L
TSS	-	20	TO	1,700	MG/L
Tot. Sol.	-	150	TO	2,300	MG/L
Vol. Tot. Sol.	-	15	TO	820	MG/L
pH	-	4.9	TO	8.7	
Settl. Sol.	-	2	TO	1,550	ML/L
Org. N	-	1.5	TO	33.1	MG/L
NH_3N	-	0.1	TO	12.5	MG/L
Sol. PO_4	-	0.1	TO	6.2	MG/L
Tot. Coli.	-	20,000	TO	90×10^6/100	ML
Fec. Coli.	-	20,000	TO	17×10^6/100	ML
Fec. Strep.	-	20,000	TO	2×10^6/100	ML

As mentioned, urban stormwater in itself is a significant contributor to the problem since it picks up a variety of known and unknown pollutants as it drains from urban land area. Figure 3 illustrates some selective data on urban stormwater characteristics. As noted, the extremely high chlorides concentrations have been attributed to deicing salts. Our program has done some work in this area resulting in the following conclusions:

1. Highway salts can cause injury and damage across a wide environmental spectrum.
2. Practically all highway authorities in the U.S. believe that ice and snow must be removed quickly from roads and highways and that "bare pavement" conditions are necessary, often resulting in excessive salt application.
3. Salt storage sites are persistent and frequent sources of ground and surface water contamination and vegetation damage.
4. The special additives, e.g., chromates and cyanides, found in road deicers provoke great concern because of their severe latent toxic properties and other potential side effects.
5. A sufficient number of incidents and detailed studies have been described to show adverse impact of deicing salts to water supplies and receiving waters.
6. In less severe cases as salt intrusion into public water supplies--salt free patients have been cautioned to change their potable water source.
7. Deicing salts are found in high concentrations in highway runoff.
8. Surveillance data is needed to clearly define the many influences of deicing salts upon the environment.
9. The majority of in-depth studies support the finding that deicing salts are a major factor in vehicular corrosion and roadway damage. The literature also indicates that rust

FIGURE 3

CHARACTERISTICS OF URBAN STORMWATER
(Selected Data)

BOD_5	-	1	to	>700	mg/L
COD	-	5	to	3,100	mg/L
TSS	-	2	to	11,300	mg/L
Tot. Sol.	-	450	to	14,600	mg/L
Vol. Tot. Sol.	-	12	to	1,600	mg/L
Settl. Sol.	-	0.5	to	5,400	mL/L
Org. N	-	0.1	to	16	mg/L
NH_3N	-	0.1	to	2.5	mg/L
Sol. PO_4	-	0.1	to	10	mg/L
Tot. PO_4	-	0.1	to	125	mg/L
Chlorides	-	2	to	25,000	mg/L*
Oils	-	0	to	110	mg/L
Phenols	-	0	to	0.2	mg/L
Lead	-	0	to	1.9	mg/L
Tot. Coli.	-	200	to	146×10^6	/100 mL
Fec. Coli.	-	55	to	112×10^6	/100 mL
Fec. Strep.	-	200	to	1.2×10^6	/100 mL

*With highway deicing

inhibiting additives do not produce results to justify their continued use. It is further noted that deicers may attack and cause damage to telephone cables, water distribution lines and other utilities adjacent to streets and highways.

10. There is little doubt that road deicers can disturb a healthy balance in soils, trees and other vegetation comprising the roadside environment.

Sewer Separation

When considering combined sewer overflow problems, first attention is generally given to the construction of separate sanitary and storm sewer systems. In contrast, the 1964 PHS study stipulated that alternative solutions be investigated to determine if means other than sewer separation could be found at lower cost.

The previously mentioned APWA study of combined sewer problems indicated that if all communities with combined sewers in this country were to effect sewer separation, they would face an expenditure of approximately 85 billion dollars at today's cost. Of this amount New York State's share would roughly come to $18 billion, the highest figure for any state in the nation. It was further estimated that the use of alternate measures could reduce the national figure to about 25-30 billion dollars.

It is again emphasized that urban stormwater runoff itself can be a significant source of stream pollution. Sewer separation would not cope with this pollution load. An EPA study revealed that if separation were used, the reduction in wet-weather pollution would be only 50 percent. The other 50 percent would remain in the untreated urban storm runoff.

IV. CORRECTIVE METHODS

Program research, development and demonstration projects have provided significant results, and have illustrated that sewer separation in most cases is not the logical course of action. We have categorized three basic approaches other than separation: control, treatment and combinations of the two.

Control

Control of combined sewer overflows can be obtained by reduction or equalization of peak stormwater flows, increasing the effective capacity of the sewerage system, minimizing infiltration and by source prevention techniques.

For existing system control, the operator can attempt to maximize wastewater treatment at the sanitary plant during wet-weather by trying to contain as much flow or treat as much sewage as possible during a storm flow occurrence. This would serve to reduce wet-weather by-passing which at the beginning of storm flow can have a high pollutant concentration, as previously described. It is recognized this extra plant burden may decrease treatment efficiencies somewhat, and create added sludge or solids handling problems; however, these practices for only short periods during storm flows are well worth the effort. If the operator determines that hydraulic loading will cause a serious upset of a unit process then primary treatment plus disinfection should be considered as a minimum measure.

In Detroit, where the prevailing direction of storms is known, the operator receives advanced information on storms from a remotely stationed rain gauge. The treatment plant pumping is increased, thus lowering the surcharged interceptor gradient, allowing for greater interceptor storage capacity and conveyance. This practice has enabled the city to entirely contain and treat many intense spot storms plus many scattered city-wide rains.

The operator should also concern himself with improved regulator inspection and maintenance, and preventive schedules to minimize the occurrence of overflows. Overflows during dry as well as wet-weather due to malfunctioning devices and clogged orifices can thus be alleviated. Tide gate conditions allowing backwater intrusion can be corrected, and diversion structure settings can be raised to obtain more interceptor carrying capacity.

Municipalities can also control combined sewer overflows without large and costly modifications by concerning themselves with infiltration and extraneous inflow. Excess flow caused by infiltration is a major thief of capacity that would otherwise be available to transport wastewater and can thereby affect proper operation of sewerage systems and, consequently, the quality of streams. Other adverse impacts caused by infiltration include: (a) surcharging and back-flooding into streets and private areas and need for relief sewers ahead of schedule; (b) surcharging of treatment plants and pumping stations, causing flow by-passing, decrease in treatment efficiency, and higher treatment costs; and (c) diversion of raw wastewater and greater incidence and duration of overflows. The APWA has reported that infiltration was a pronounced problem during dry weather in 14 percent of communities surveyed and in 53 percent of the communities during wet weather. The APWA also indicates that other sources of extraneous inflow compounding the problem include roof leaders; depressed manholes covers; cellar, foundation. and yard drains; air conditioning and industrial cooling waters; and other connections.

Control of infiltration should first take place during sewer pipe installation. Better construction materials and proper installation techniques are necessary. The new methods of sewer sealing and lining should be fully evaluated before major rehabilitation or replacement is undertaken.

Infiltration surveys should be undertaken when extraneous inflows are suspected. Such surveys may use television and other visual pipeline inspection, smoke tests, air and water pressure tests, and various flow techniques. Undue deposits, partial blockages and cave-ins causing premature surcharging and dry- and wet-weather overflows (usually in older sewer systems) will also be pinpointed for subsequent corrective action.

Building connections to street sewers are a major source of infiltration. As much as 70 to 80 percent of the infiltration load can occur in these lines. Accordingly, the aforementioned infiltration control practices should be strictly followed here.

Before a municipality considers removing extraneous inflows, the following basic factors should be considered:

1. Determination of what a "clean" or unpolluted inflow really is. For instance, subsurface drainage may be contaminated leachate or contain toxic material washed from basement floors.
2. Sewer septicity and odor conditions that may arise because of lowered flow from the elimination of long-standing inflow sources.
3. Effect on the public of any sudden decision to eliminate inflow sources and the associated problems of enforcement.
4. The strong possibility that communities will be forced to treat separate urban runoff sometime in the future indicates that the reconnection of certain so-called "clean" waters from sanitary to storm drains may be done in vain.

Studies have indicated that it may be cheaper to remove solids from the street surfaces by sweeping than by eliminating them via the sewer system. One set of figures showed that street sweeping costs $24 to $30/ton of solids removed as compared to $60 to $70/ton of solids removed via the sewerage system. What may be even more important is that the wet-weather overflow polluting potential of these solids is eliminated by the urban surface removal practice.

Aside from abating the usual contaminants, a particular advantage of effectively removing the dust and dirt fractions prior to sewer entry would come from the reduction of major amounts of the more exotic pollutants which include heavy metals (lead, zinc, cadmium, mercury, copper, chromium), pesticides and PCB's, and nutrients that commonly adhere to the surfaces of solids. Because of the potential land and groundwater contamination, care should be given to the solids disposal site selection and the fate and effects of these pollutants. At this juncture it is appropriate to mention that greater efforts should be applied in the area of non-routine stormwater constituents. Their impacts and abatement measures must be further researched, whether they be by surface "housekeeping" at the source or treatment of the storm flow itself.

It is recommended that the newer and more promising street cleaning equipment such as vacuum sweepers, air brooms and wet scrubbers be further evaluated and employed as opposed to conventional sweeping and flushing methods. The newer devices offer benefits in picking up the dust and dirt particles rather than redistributing them for aesthetic purposes as the conventional devices do.

Certain land use, zoning, and construction site erosion control practices are other ways of alleviating the solids burden to the receiving streams or treatment plants by surface source prevention.

Cleansing of catch basins, sewer lines, wet wells and other appurtenances by flushing or dry mechanical means may reduce solid loadings in wet-weather discharges and alleviate premature overflows during dry or wet periods due to partial or complete sewer obstructions. But here we must weigh the benefits of system cleaning against "closing the loop" by the installation of wet-weather flow control and/or treatment facilities.

It is emphasized that before a community considers the establishment or continuation of the household garbage grinding practice, it must be realized that increased solids deposition in both combined and sanitary sewer lines will occur at times of low flow during dry weather which will be scoured out by the high storm flow conditions. As a result, the overflows will create more severe stream impacts. The jurisdiction's plans regarding future overflow control and treatment will be an important consideration since again the "loop" will be closed.

If there is insufficient carrying capacity in the sewer lines, polymer addition may serve to measurably reduce fluid friction. Research has shown that polymeric injection can increase flow capacity as much as 2.4 times at a constant head. This method can be used as a measure to correct troublesome pollution-causing conditions such as localized flooding and excessive overflows. Preliminary cost comparisons have shown this procedure to be feasible.

Advanced Control Systems

In this segment of the talk, some of the newer and more advanced technology being developed by our Program will be described.

Flow Regulation

Several methods have been used to reduce operation problems associated with the conventional regulator devices. Cincinnati utilizes telemetered monitoring to detect unusual or improper dry-weather overflows. More sophisticated approaches are being applied by the Minneapolis-St. Paul Sanitary District and the Cities of Detroit, and Seattle. All three jurisdictions are making use of unused storage capacity within the existing sewerage system for the purpose of reducing the frequency and volumes of overflows. For instance, in the period from 1969 to 1970, Minneapolis was able to reduce overflow occurrences by 55% and the volume of overflow by 85%. The general approach comprises remote monitoring of rainfall, flow levels, and sometimes quality, at selected locations in the network, together with a centrally computerized control console for positive regulation of the overflow structures. Figure 4 depicts the computer console and strategy room in Seattle, and is a preview of what the operator in 1980 may be contending with.

New types of regulators such as positive control gates and inflated rubberized-fabric dams (Figure 5) have been demonstrated successfully. Another unique overflow device which has been constructed for full-scale demonstration utilizes fluidic technology; and requires no moving parts or external power since operation is entirely dependent upon motion of the wastewater. Improved regulator capability and reduced operation and maintenance costs are anticipated. Additional improvement in regulators is now in progress.

Storage

Storage offers direct control by containing the wastewaters produced during wet-weather periods. The use of storage facilities for controlling combined sewer overflows has been convincingly demonstrated. The general procedure involves the return of re-

Figure 4. Computer console for augmented flow control system, Seattle, Washington.

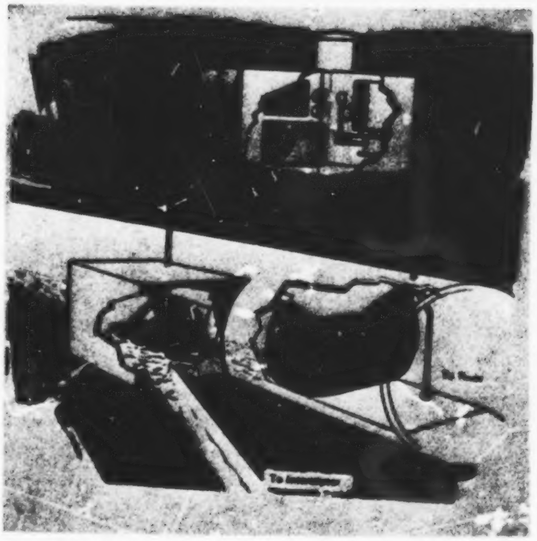


Figure 5. Inflatable Control Gate System

tained overflows to the conventional treatment works for subsequent treatment during low flow, dry-weather periods.

Concrete holding tanks are the most commonly used type of storage facility. The storm stand-by tanks at Columbus, Ohio, shown in Figure 6, constructed as early as 1932, were recently modernized by installation of sludge collection and automatic flow control equipment. The City of Boston has commenced operation of overflow holding tanks designed to provide 10-minute settling plus chlorination for treating excess overflows of 233 million gallons per day. New York City and Milwaukee have similar facilities in operation. The New York City plant has four storage tanks which have a combined capacity of 9.7 million gallons. Intercepted storm flow is stored, degritted, and pumped, along with the sludge back to a nearby Municipal Treatment Plant. Excessive overflows receive treatment by sedimentation through the tank and are chlorinated and discharged. The objective of the facility is to reduce coliform and solids contamination of Jamaica Bay.

Chippewa Falls, Wisconsin has constructed an asphalt-lined basin providing storage for up to 3.5 million gallons of overflow. (Figure 7) During the 1969 - 1970 evaluation period, 50 river discharges out of 62 storm overflows were eliminated.

Two basic problems encountered by conventionally-designed storage facilities in urban areas are land cost and availability, and adverse aesthetic impacts. In this regard, we are seeking new concepts. A major demonstration in Chicago involves the new concept of "deep tunnels". The cost of the Metropolitan Chicago tunnel storage system is estimated at over one billion dollars as contrasted to over four billion for sewer separation. Additional benefits of tunnel (or in-sewer) storage are a result of coverage of an expanded area or length. Thus, storage is more readily avail-

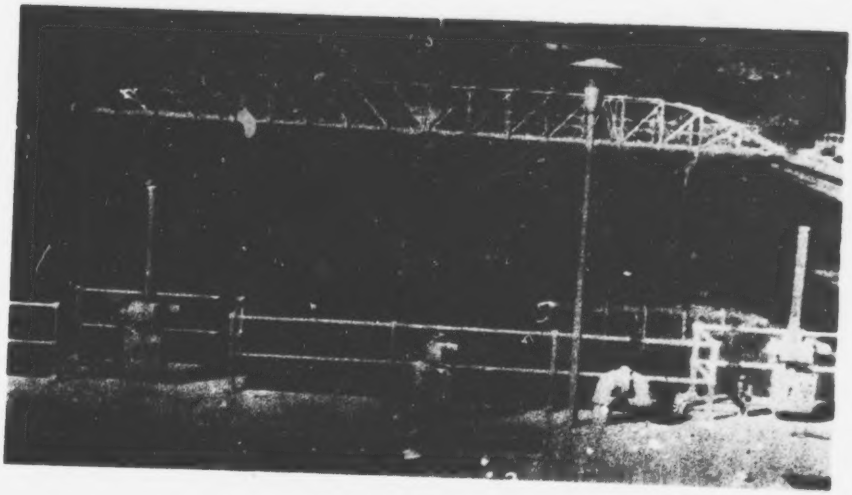


Figure 6. Storm Stand-by tank with upper portion of sludge collection mechanism visible, Columbus, Ohio.

Figure 7. Asphalt-lined basin providing storage for up to 3.5 MG, Chippewa Falls, Wisconsin.

able to remote areas, hydrographs may be smoothed or reduced for treatment facility design because intense storms often are quite localized, and overflows greater than storage capacity can be selectively and automatically discharged to the most suitable stream locations. Another subsurface storage idea to be demonstrated in Lancaster, Pa., is the underground "silo". The use of a 50-foot diameter, 100-foot deep silo could afford over 1 million gallons of storage. The preliminary design is shown in Figure 8.

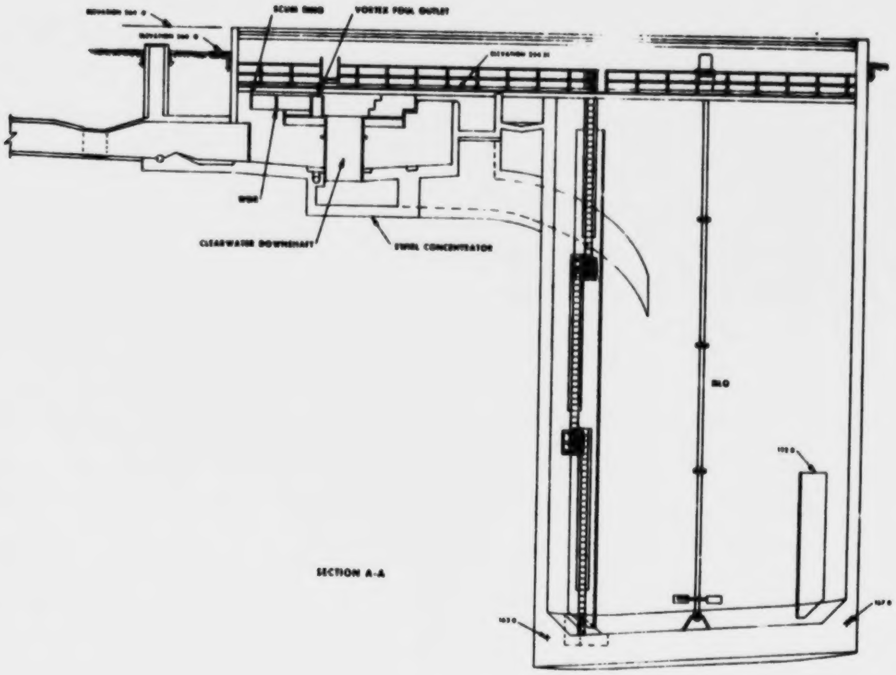
Other designs requiring little or no urban land include offshore storage and the use of natural underground formations. Two demonstration projects have evaluated the use of flexible neoprene-coated nylon fabric material as underwater containers, for the temporary storage of combined sewer overflows. Figure 9 presents a drawing of such an installation.

The engineer and operator will be interested in the sludge-handling aspects of temporary storage. Two possibilities are the re-suspension of solids by agitators and settling prior to pump-back. Re-suspension can provide easier draw-off and is being evaluated. However, if sludge is settled, on-site sludge disposal in lieu of solids pumped back in stored flow should be considered.

Design criteria should be based on the pollution abatement results expected. For example, Milwaukee used a mathematical model to determine size and projected efficiency of its holding tanks.

Wherever possible, design of full-scale facilities should consider the total environmental impact, including aesthetics. Figure 10 is a conceptual drawing showing an off-shore site in Lake Erie at Cleveland, Ohio

A concept worthy of note, which was successfully demonstrated in


SCUM RING
VORTEX FOUL OUTLET
WEIR
CLEARWATER DOWNSHAFT
SWIRL CONCENTRATOR
SECTION A-A

Preliminary Drawing - Elevation View of System, Lancaster, Pa.

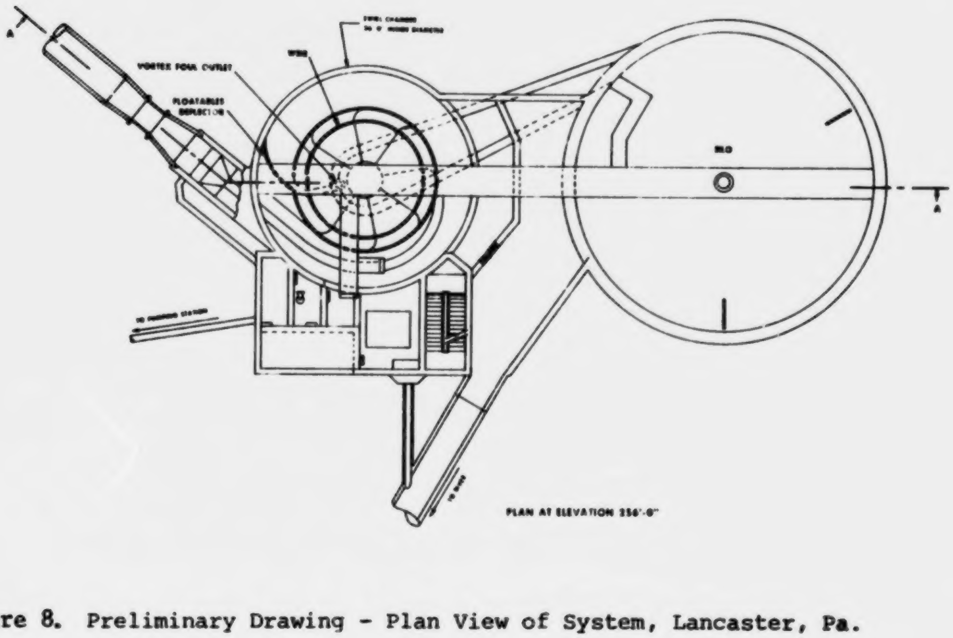
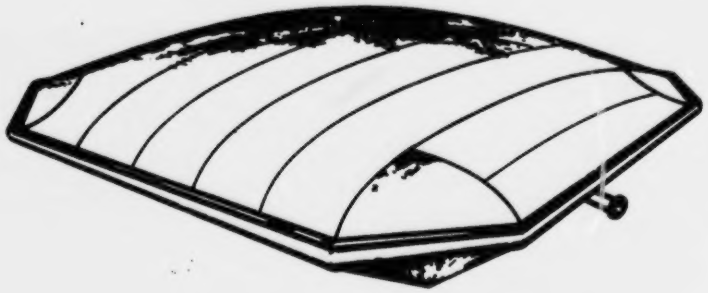

A
VORTEX FOUL OUTLET
FLOATABLES DEFLECTOR
WEIR
SWIRL CHAMBER
A
PLAN AT ELEVATION 256'-0"

Figure 8. Preliminary Drawing - Plan View of System, Lancaster, Pa.

FIGURE 9

UNDERWATER TANK



INTERIOR SECTIONED VIEWS

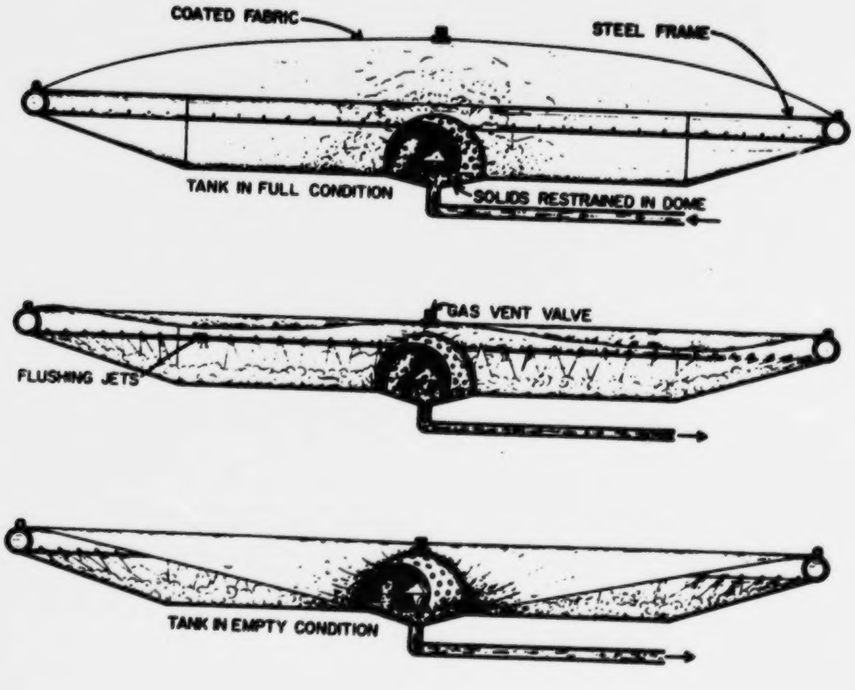

COATED FABRIC
STEEL FRAME
TANK IN FULL CONDITION
SOLIDS RESTRAINED IN DOME
GAS VENT VALVE
FLUSHING JETS
TANK IN EMPTY CONDITION

Figure 10. Conceptual design of combined sewage retention-stabilization basin, Cleveland, Ohio

London, England and Decatur, Illinois, is the conversion of existing or abandoned sanitary treatment units, in this case sedimentation tanks, to storm holding facilities as part of a plant expansion. Also, plans have been proposed to use an abandoned trickling filter as a storage tank for stormwater infiltration.

Porous Pavement

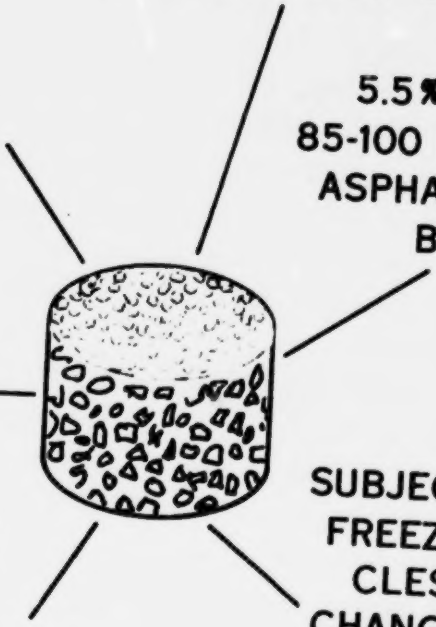
Another feasible method to attenuate flows is the installation of porous pavement. This pavement is made of asphaltic-concrete and has been developed for an ability to allow 60 or more inches per hour of rainfall to permeate through its depth (Figure 11). If used for major highway, street, and parking lot paving projects, it would have the potential for reducing capacity and associated costs for both sewer and wet-weather flow treatment systems, a feature attributable to the porous pavement's ability to equalize flows entering or divert flows away from the sewerage system. This type of pavement installation can also offer a substantial benefit by recharging water supplies. Even more important are the safety features which could be realized, i.e., an increased coefficient of friction which will help prevent wet skidding or hydroplaning accidents, and enhanced visibility of pavement markings due to more rapid removal of rainwater and rougher surfaces. However, when porous pavement is considered, we must realize that such features as geographical area, temperature, subsurface soil condition, and the possibility of groundwater contamination may play an important part in design and site selection.

New Sewer Systems

New types of sewer systems being demonstrated, based on vacuum and pressure operation for the collection and conveyance of sanitary sewage, can reduce the waste volume generated, reduce conduit sizes, eliminate infiltration, minimize associated installation and treatment costs,

FIGURE 11

POROUS ASPHALTIC CONCRETE FEATURES



AGGREGATE GRADED TO ALLOW A WATER FLOW OF 76"/HOUR

EXCEEDED THE MINIMUM MARSHALL STABILITY CRITERION FOR MEDIUM TRAFFIC USES

5.5% BY WT. OF 85-100 PENETRATION ASPHALT CEMENT BINDER

AEROBIC ACTIVITY UNDER PAVEMENT NOT. IMPAIRED

DURABILITY TEST INDICATED THAT HEIGHTENED EXPOSURE TO AIR OR WATER DID NOT PRODUCE ASPHALT HARDENING

SUBJECTED TO 265 FREEZE-THAW CYCLES WITH NO CHANGES IN PHYSICAL DIMENSIONS, MARSHALL STABILITY VALUES OR FLOW RATES and alleviate overflows. More on this subject will be presented later by Mr. Carcich.

Treatment

Other than waste storage, it may be considered necessary that combined sewer overflows be treated, either at the individual outfall locations or at a centralized facility. Centralization, although offering benefits in reduced plant costs, invariably requires high expenditures for the installation of large, combined sewer transport conduits or interceptors.

Conventional treatment processes apply basically to the relatively steady-state conditions of sanitary sewage, whereas combined sewer overflows occur on an intermittent and random basis. Following rainfall, these flows exhibit highly-varying patterns in both quality and quantity over short periods of time. Consequently, it has been difficult to directly adapt existing treatment methods to storm generated overflows, especially the micro-organism dependent biological processes. Adverse flow conditions and unpredictable shock loadings make it advisable to consider the newer chemical and physical treatment techniques and the incorporation of automated control into the intended storm treatment facility to achieve optimum efficiency.

Rather than independent units, biological treatment systems are applied by our Program as auxiliary facilities at the conventional sewage plant for treating excess flows. This affords a viable mixed liquor and zoogloeal film. Two such biological treatment processes are on-going EPA projects. One uses activated sludge to treat overflows at Kenosha, Wisconsin. The other, at New Providence, New Jersey, utilizes plastic media and compares this with standard rock media in high-rate trickling filters for treating sanitary wastewater with a high degree of infiltration.

Treatment methods which have been evaluated or are currently under investigation by the Storm and Combined Sewer Pollution Control Program include:

1. Fine-mesh screening and microscreening
2. Dissolved-air flotation
3. Rotating biological contactors
4. High-rate plastic and rock media trickling filters
5. High-rate, single, dual and tri-media filtration
6. Swirl and helical separators
7. Advanced disinfection methods, e.g., high-rate application, on-site generation, automated operation, ozonation, and use of combined halogens (chlorine and iodine) and chlorine dioxide
8. Tube settlers
9. Powdered and granular activated carbon adsorption
10. Polymer and other chemical additives for improved settling, microscreening, filtration and flotation
11. Chemical oxidation
12. In-line or in-sewer treatment
13. Sludge handling and treatment
14. Regeneration of carbon and coagulants, and
15. Reclamation and reuse.

Time does not allow a detailed discussion of each of these methods Some of the more promising treatment techniques will be discussed.

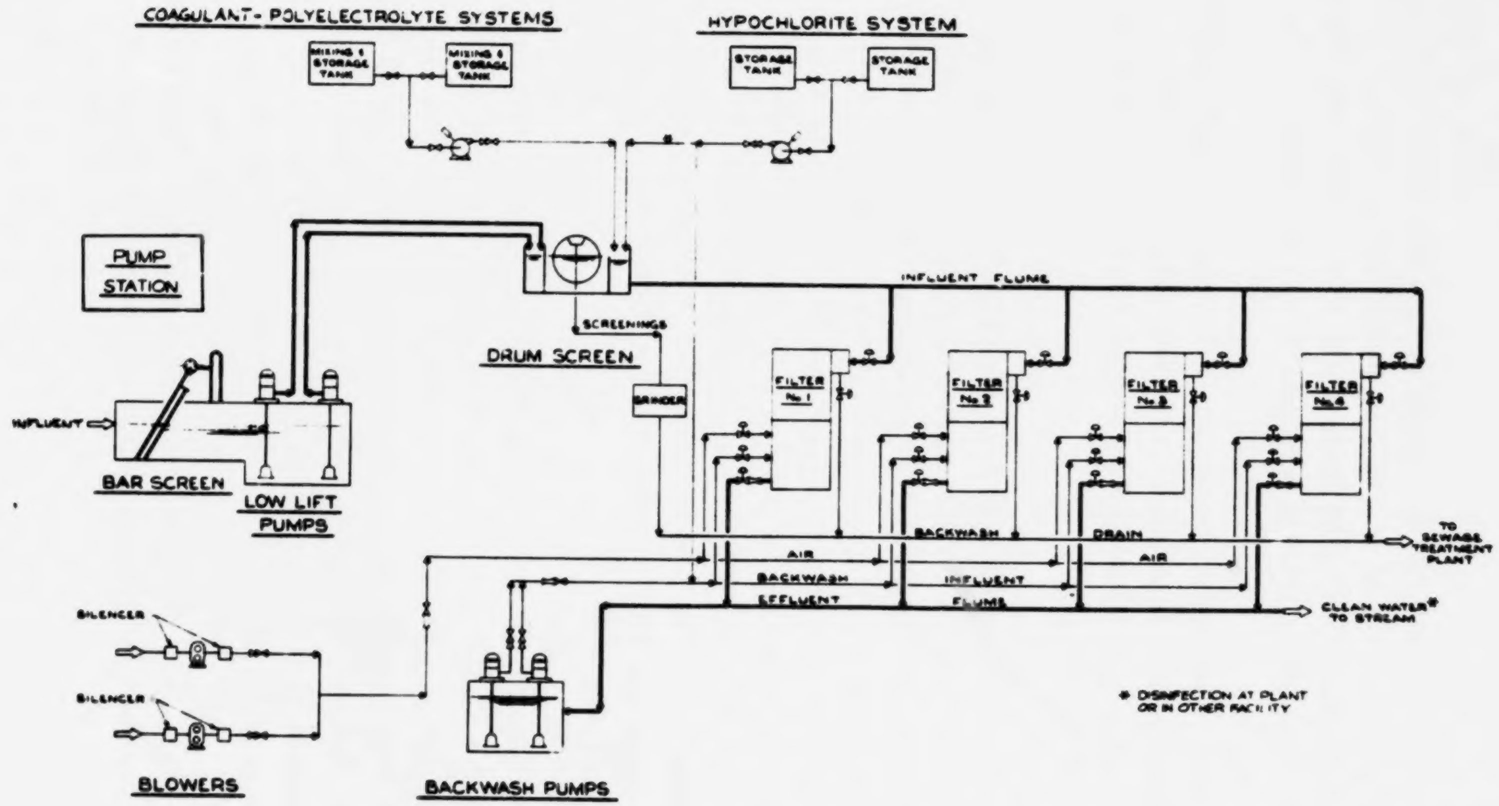
Since high throughput rates are necessary for combined sewer overflows, the sanitary treatment processes are being studied for possible modifications. For example, the microstrainer is conventionally designed for polishing secondary sewage plant effluent at an optimum rate of around 10 gallons per minute per square foot. Tests on a pilot microscreening unit in Philadelphia, Pa. have shown that, at high flux rates of 35 to 45 gpm/ft^2, suspended solids removals in combined overflows exceeding 99 percent can be achieved. Mr. George Glover will speak about this in more detail this afternoon.

Increased flow rates greatly reduce capital costs and space requirements. Increased throughputs have also been obtained with other fine-mesh screening processes, for example, fiberglass filtration and dissolved-air flotation.

An EPA study in Cleveland showed high potential for treating combined sewer overflows by contact coagulation and ultra high-rate filtration. Figure 12 depicts the process flow diagram. With the high loadings of 16 to 32 gpm/ft^2 surface area, removal of solids is effectively accomplished throughout the entire depth of filter column. Test work showed suspended solids removal up to and exceeding 90 percent and BOD removals in the range of 60 to 80 percent. Substantial reductions, in the order of 30 to 80 percent of phosphates, can also be obtained. Mr. Pat Harvey will discuss this at length later on today.

Results from a 5.0 MGD screening and dissolved-air flotation demonstration pilot plant, in Milwaukee, indicate that greater than 70 percent removals of BOD and suspended solids are possible. Findings also revealed 85 to 97 percent reduction in suspended solids, and better than 90 percent reduction in phosphate can be achieved as an additional benefit, by employing chemical coagulants. Mr. Gupta will give his presentation on this topic this afternoon.

A unique variation of the usual coagulation-adsorption, physical-chemical treatment process has been demonstrated in Albany. This


COAGULANT - POLYELECTROLYTE SYSTEMS
MIXING & STORAGE TANK
MIXING & STORAGE TANK
HYPOCHLORITE SYSTEM
STORAGE TANK
STORAGE TANK
PUMP STATION
INFLUENT
BAR SCREEN
LOW LIFT PUMPS
DRUM SCREEN
SCREENINGS
GRINDER
INFLUENT FLUME
FILTER No. 1
FILTER No. 2
FILTER No. 3
FILTER No. 4
BACKWASH
DRAIN
AIR
BACKWASH
EFFLUENT
AIR
INFLUENT
FLUME
TO SEWAGE TREATMENT PLANT
CLEAN WATER* TO STREAM
* DISINFECTION AT PLANT OR IN OTHER FACILITY
SILENCER
SILENCER
BLOWERS
BACKWASH PUMPS

HIGH RATE FILTRATION INSTALLATION PROCESS FLOW DIAGRAM

FIGURE 12

system, shown schematically in Figure 13, is comprised of a 100,000 GPD trailer mounted pilot plant where both powdered carbon and coagulants are added in a static mixing-reaction pipeline, and the resultant coagulated matter is flocculated downstream, separated by tube-settlers and polished by multi-media filtration. The project also demonstrated regeneration of alum and activated carbon by fluidized-bed incineration.

At this point it is appropriate to bring out an important fact of which future designers of storm overflow treatment facilities must be cognizant--process efficiency should not be considered in the usual terms of percent removal used in municipal treatment. It was found during the microstrainer and dissolved-air flotation operation that, due to extreme variation of the influent suspended solids concentration, removal efficiency would also vary while the more desirable effluent concentration remained relatively constant. For example, a typical effluent concentration of 10 mg/l suspended solids would yield a reduction of 99 percent for an influent concentration of 1,000 mg/l, whereas the suspended solids reduction would be only 50 percent if the influent concentration were 20 mg/l. This phenomenon is apt to reoccur in other physical-chemical stormwater treatment operations.

Another project has studied a new biological process, described as the rotating biological contactor consisting of a series of shaft-mounted rotating disks. Similar in principle to trickling filtration, a biological growth attaches onto the disks. Under steady loading rates, efficiencies exceeding those of the trickling filter have been attained, but a surge tank appears essential. Figures 14 and 15 give a close-up of the rotating disks and an overall view of the pilot facility, respectively.

Another approach in overcoming the extreme variation in overflow rates is to provide surge facilities prior to the storm treatment plant or

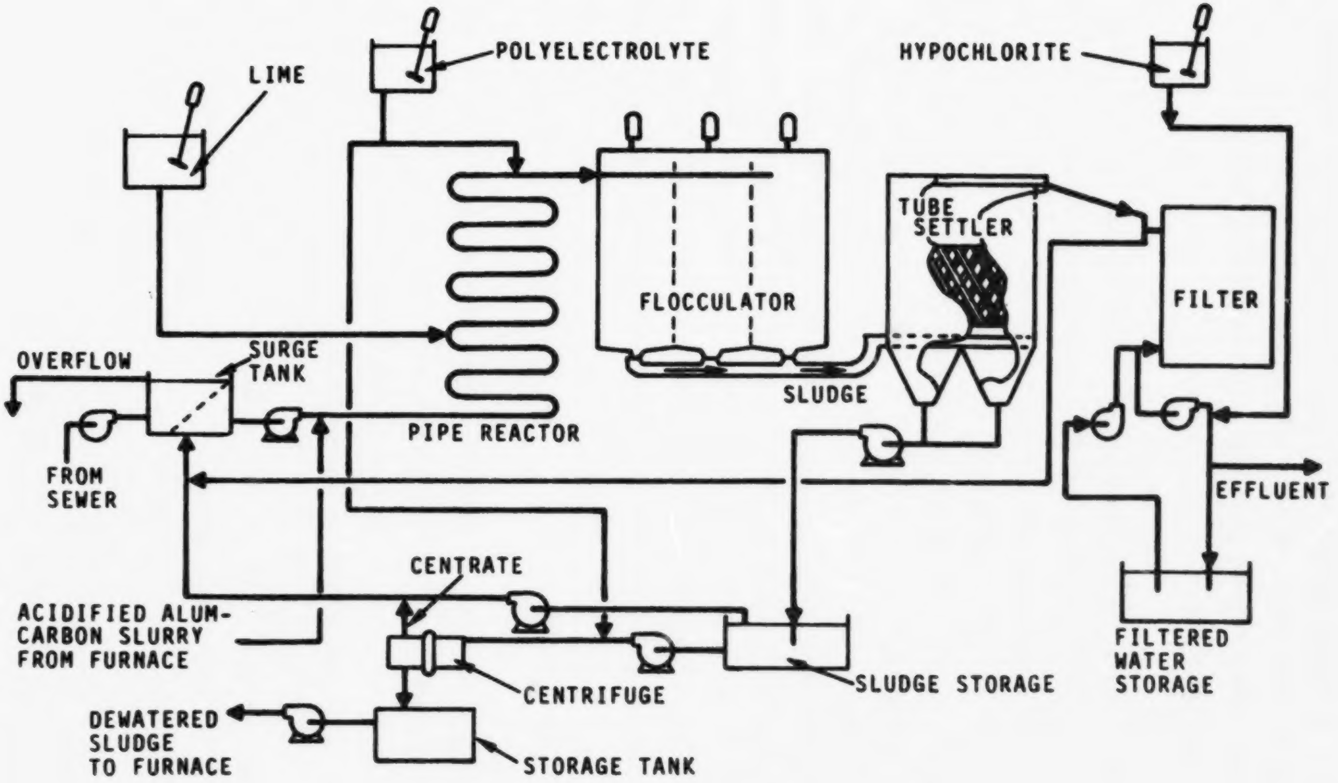

LIME
POLYELECTROLYTE
HYPOCHLORITE
FLOCCULATOR
TUBE SETTLER
FILTER
OVERFLOW
SURGE TANK
FROM SEWER
PIPE REACTOR
SLUDGE
EFFLUENT
CENTRATE
ACIDIFIED ALUM-CARBON SLURRY FROM FURNACE
CENTRIFUGE
SLUDGE STORAGE
FILTERED WATER STORAGE
DEWATERED SLUDGE TO FURNACE
STORAGE TANK

Figure 13. SCHEMATIC FLOWSHEET OF MOBILE PILOT PLANT

Figure 14. Close-up view of rotating biological disks, Milwaukee, Wisconsin.

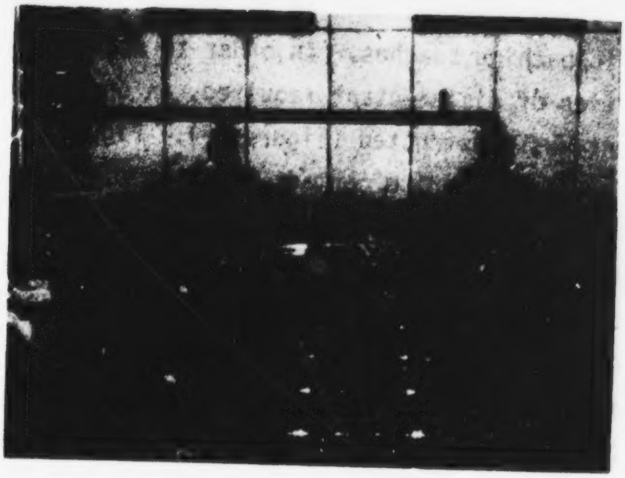


Figure 15. Overall view of rotating biological disks, Milwaukee, Wisconsin.

the municipal plant. The surge basin(s) (or existing combined sewers) could furthermore serve a dual function in equalizing not only wet-weather flows but dry-weather flows as well. In this way, a single future treatment system can readily be designed for storm and sanitary flow conditions. This could also assist presently overloaded sanitary plants in obtaining more uniform operation. Short-term storage incorporated into the treatment plant would even out the daily cycle of dry-weather flows allowing for more efficient use of the treatment process over the entire 24 hours. Equalization would permit reduced treatment process design capacity. Further analysis is necessary to determine the most economical break-even point between the amount of storage versus the treatment capacity. The designer should recognize the wet-weather treatment plant's capability to draft stored flow continuously while it is raining in his evaluation of the optimum surge-treatment system.

New Orleans has demonstrated the use of sodium hypochlorite for disinfection of storm flows as high as 11,000 cfs, to both reclaim and protect public bathing beaches. In order to economically provide the large quantities of disinfectant required, an on-site hypochlorite batching plant was constructed (Figure 16). Figure 17 gives a view of the massive-size chlorine contact basin in operation.

The disinfection of combined sewage entails certain differences, which make the design and operation of facilities difficult when compared to sanitary sewage. The highly varying qualitative and quantitative character of the storm generated inflows require disinfectant dosages to be based on a predicted rather than an established technique. A decrease in temperature decreases disinfectant kill power. This points to the importance of temperature in additon to the usual (time and dosage) control parameters. As temperature is apt to have a much wider range for runoff waters than it does for domestic sewage flows, combined sewage may require disinfectant dosage to vary season-

Figure 16. Stormwater disinfection project - hypochlorite batching plant, New Orleans, Louisiana.

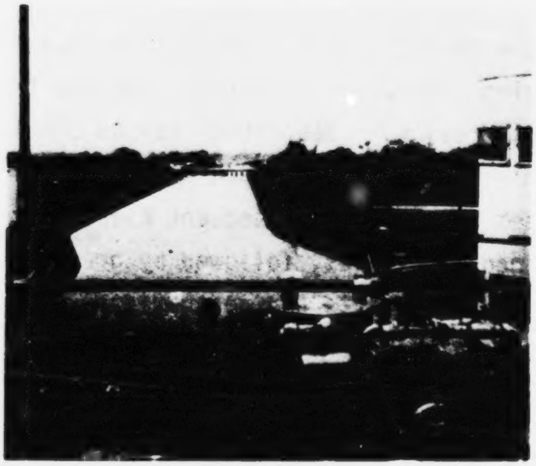


Figure 17. Stormwater disinfection project - chlorine contact basin, New Orleans, Louisiana.

ally or as effected by ambient temperature.

The Storm and Combined Sewer Overflow Technology Program is also searching for high-rate disinfection systems to save on large tankage requirements for the high storm flow rates encountered, with the help of more rapid oxidants e.g. chlorine-dioxide, and by imparting greater turbulence to the flow. Successful attempts toward high-rate disinfection are being noticed at our Philadelphia, Pa. and Onondaga County, New York demonstration sites. The Philadelphia project also made an evaluation of ozone, generated on-site for disinfection purposes. Another study proposes the use of combined halogens (chlorine and iodine) to provide more effective disinfection of viruses as well as bacteria in a swimming lake. This study also supports dechlorination by activated carbon or use of ozone, with a relatively short half life, in lieu of chlorine to alleviate residual toxicity problems to fish life. Mr. George Glover will present more on this subject.

Combinations

When a single method is not likely to produce the best possible answers to a given pollution situation, various treatment and control measures--as previously described--may be combined for maximum flexibility and efficiency. One such combination might be: in-sewer or off-system storage for subsequent overflow treatment in specifically designed facilities, followed by groundwater recharge or recovery for water sports and aesthetic purposes. Another combination might be flow retention with pump or gravity feed-back to the sanitary sewerage system.

In all cases the optimum abatement plan for stormwater overflow pollution will have to be evaluated separately for the geographical area in consideration. Aside from climatological conditions, terrain, and land uses, choice of control and treatment will depend on the existing sewerage system configuration. For example, systems with large contributory areas and few overflow points present problems and require design philosophies which differ from those in systems divided into many subdrainage areas with individual combined wastewater outfalls.

The temporary storage concept, previously discussed as a control process, also provides for a certain degree of treatment by settling, for excessive overflows greater than the design storage capacity discharging directly to the receiving stream. Likewise, this settling potential for flows less than design capacity, together with on-site solids disposal usually overlooked, should be definitely considered. The proposed prototype demonstration for Lancaster, Pennsylvania, previously cited and shown schematically in Figure 18, will pre-treat by a swirl device and microstrain and disinfect discharges greater than the storage capacity of the "silo" structure.

Mr. Clemens, Michigan installed a system involving discharge of combined sewage overflows into a series of three "lakelets" each equipped with surface aerators. Effluents pass from one pond to the next through microstrainers and filters, and the final effluent is chlorinated. This control and treatment scheme is designed to have no adverse aesthetic impacts, and the possibility of reusing these waters for recreational purposes is being explored. Figure 19 shows a schematic of the Mt. Clemens facility.

A conceptual engineering study for the Washington, D. C. area (Figure 20) has shown that it would be feasible to construct a control-treatment facility to handle combined sewer overflows up to 3,000 cfs. A 175 million gallon storage facility is tentatively planned with an overhead parking garage, coupled with a 50 MGD high rate filtration-adsorption-disinfection plant. This treatment complex is intended to produce reclaimed waters suitable for swimming, boating, and fishing.

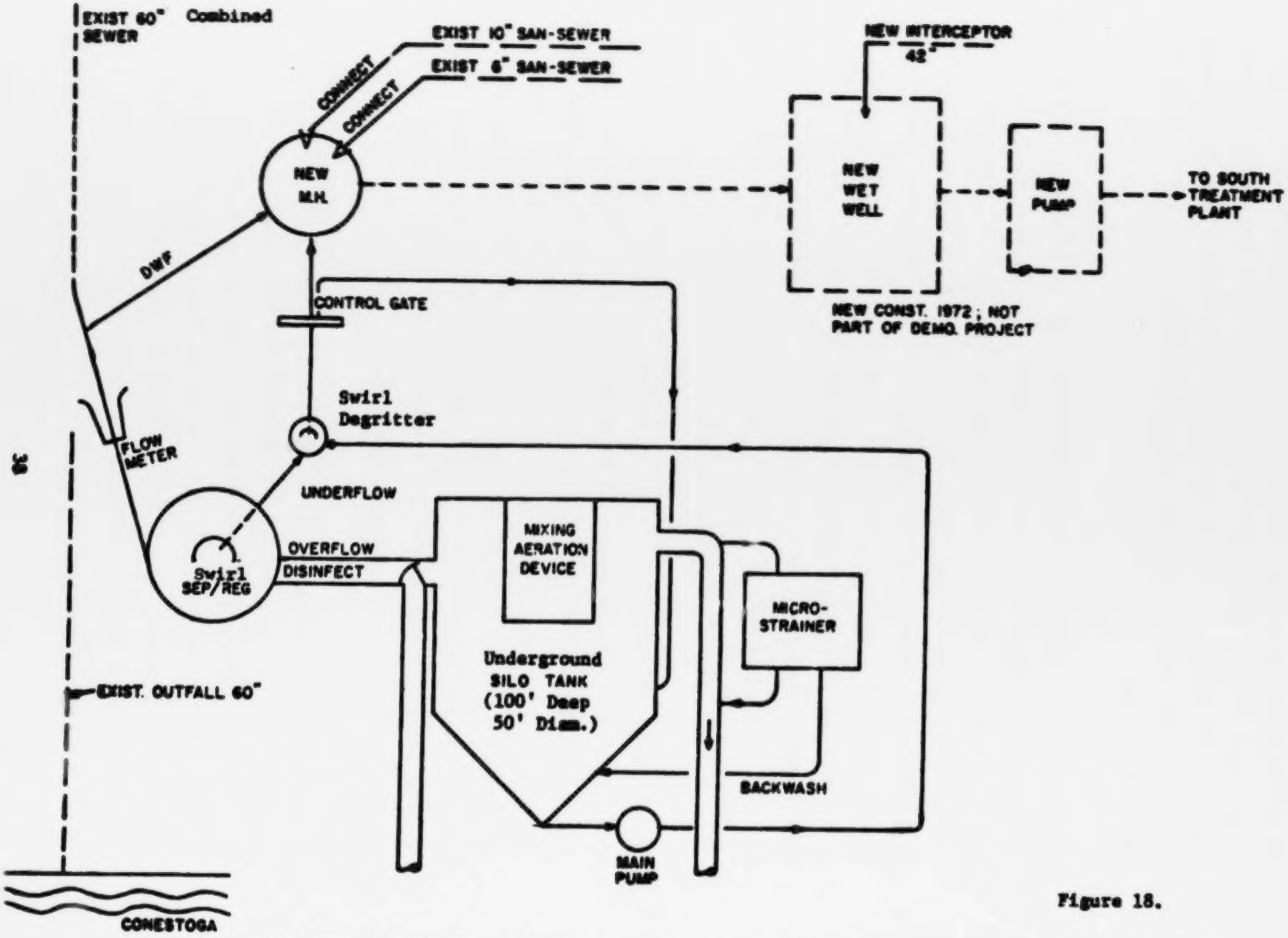
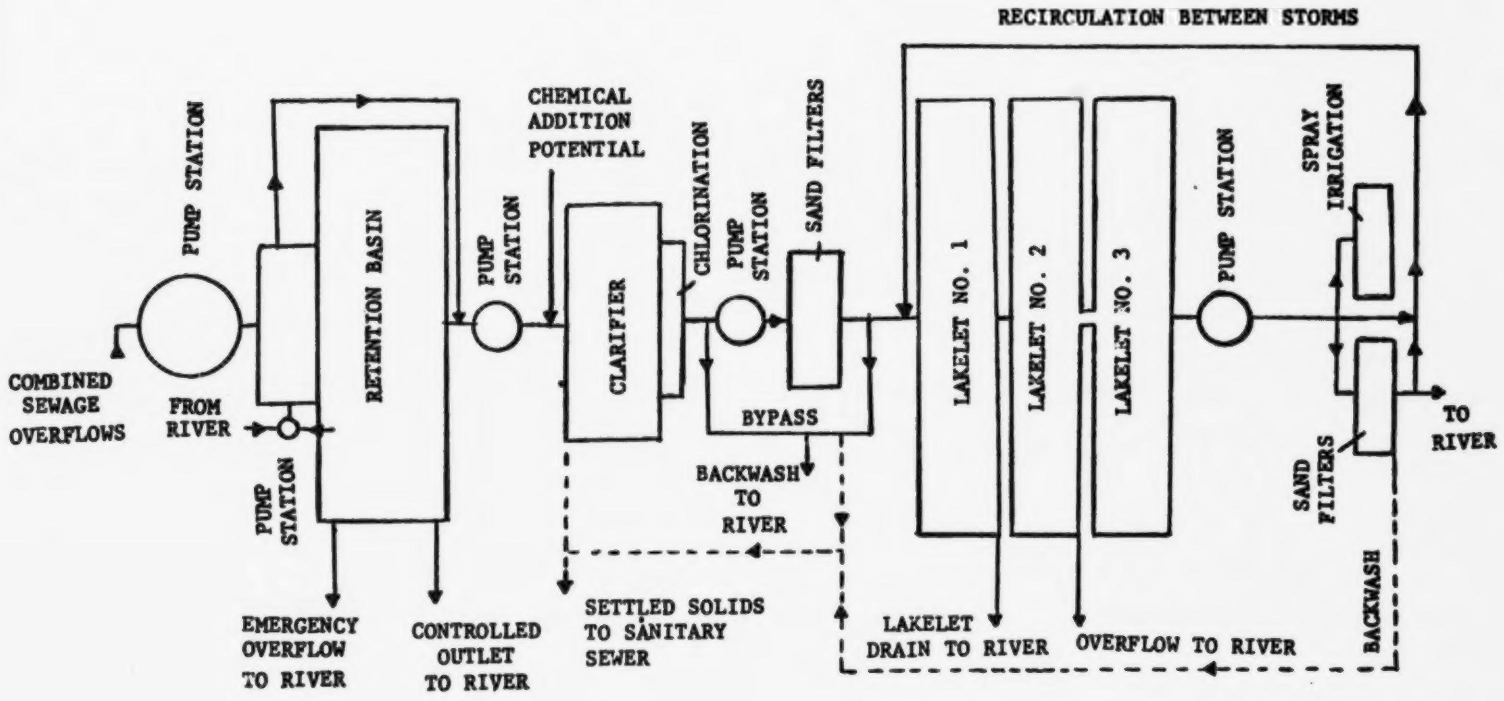

EXIST 60" Combined SEWER
EXIST 10" SAN-SEWER
EXIST 6" SAN-SEWER
CONNECT
CONNECT
NEW INTERCEPTOR 42"
NEW M.H.
NEW WET WELL
NEW PUMP
TO SOUTH TREATMENT PLANT
NEW CONST. 1972; NOT PART OF DEMO. PROJECT
DWF
CONTROL GATE
Swirl Degritter
FLOW METER
UNDERFLOW
OVERFLOW
DISINFECT
Swirl SEP/REG
MIXING AERATION DEVICE
MICRO-STRAINER
Underground SILO TANK (100' Deep 50' Diam.)
BACKWASH
MAIN PUMP
EXIST. OUTFALL 60"
CONESTOGA
River

Figure 18.

Preliminary Flow Diagram, Combined Sewer Overflow Control/Treatment System, Lancaster, Pa.


RECIRCULATION BETWEEN STORMS
PUMP STATION
COMBINED SEWAGE OVERFLOWS
FROM RIVER
PUMP STATION
RETENTION BASIN
EMERGENCY OVERFLOW TO RIVER
CONTROLLED OUTLET TO RIVER
PUMP STATION
CHEMICAL ADDITION POTENTIAL
CLARIFIER
CHLORINATION
PUMP STATION
SAND FILTERS
BYPASS
BACKWASH TO RIVER
SETTLED SOLIDS TO SANITARY SEWER
LAKELET NO. 1
LAKELET NO. 2
LAKELET NO. 3
LAKELET DRAIN TO RIVER
OVERFLOW TO RIVER
PUMP STATION
SPRAY IRRIGATION
SAND FILTERS
BACKWASH
TO RIVER

RETENTION AND TREATMENT FACILITIES
Mount Clemens, Michigan

Figure 19

FEDERAL WATER QUALITY ADMINISTRATION
KINGMAN LAKE PROJECT
WATER RECLAMATION FACILITY
PLOT PLAN

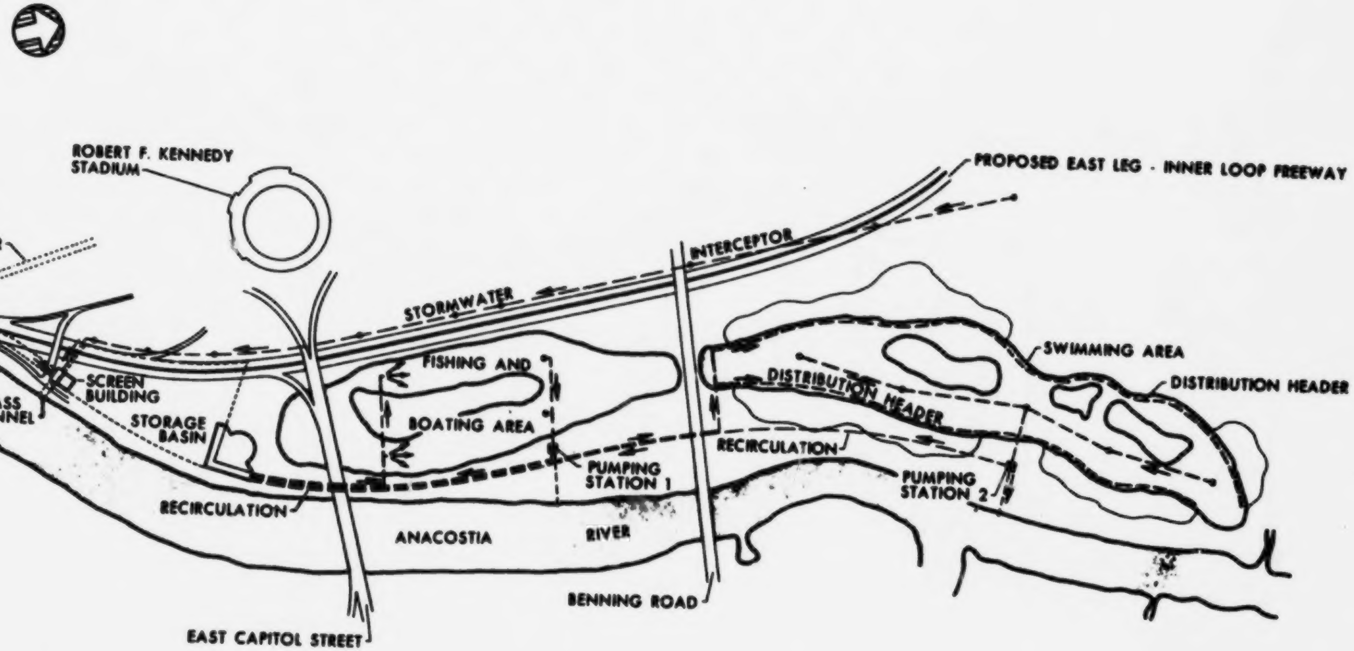

ROBERT F. KENNEDY STADIUM
PROPOSED EAST LEG - INNER LOOP FREEWAY
NORTHEAST BOUNDARY TRUNK SEWER
INTERCEPTOR
STORMWATER
INFLUENT CHANNEL
SCREEN BUILDING
BY-PASS CHANNEL
STORAGE BASIN
FISHING AND BOATING AREA
PUMPING STATION 1
RECIRCULATION
DISTRIBUTION HEADER
SWIMMING AREA
DISTRIBUTION HEADER
RECIRCULATION
PUMPING STATION 2
ANACOSTIA RIVER
BENNING ROAD
EAST CAPITOL STREET

FIGURE 20

Our Program, in conjunction with APWA, has refined and is demonstrating the swirl flow regulator/solids-liquid separator (Figure 21). The device is of simple annular-shaped construction requiring no moving parts. It provides a dual function, regulating flow by a central circular weir, while simultaneously treating combined sewage by swirl action which imparts liquid-solids separation. The low-flow concentrate is diverted to the sanitary sewerage system, and the relatively clear liquid overflows the weir into a downshaft and receives further treatment or is discharged to the stream. This device is capable of functioning effectively over a wide range of combined sewer overflow rates having the ability to effectively separate settleable and light-weight organic suspended matter at a small fraction of the detention time required for conventional sedimentation. For these reasons serious thought is now being given to the use of swirl units in series and in parallel solely as wet-weather treatment plant systems. A helical or spiral type regulator/separator has also been developed based on similar principles as the swirl device, and we are looking for further refinement. Mr. Richard Sullivan will speak on this subject following my presentation.

Flow Measurement

The quantitative and qualitative measurement of storm overflows is essential for process design, control, and evaluation. The "urban intelligence systems" previously mentioned require real-time data from rapid, remote sensors in order to achieve remote control of a sewerage network. Conventional flow meters have not been developed for the highly-varying surges encountered in combined sewers. Here, a measuring device may be subjected to very low flow rates, submergence, reverse flow, and surcharge, all during a single rainstorm. These severe flow conditions rule out the reliable and accurate application of conventional devices, such as weirs and flumes at many

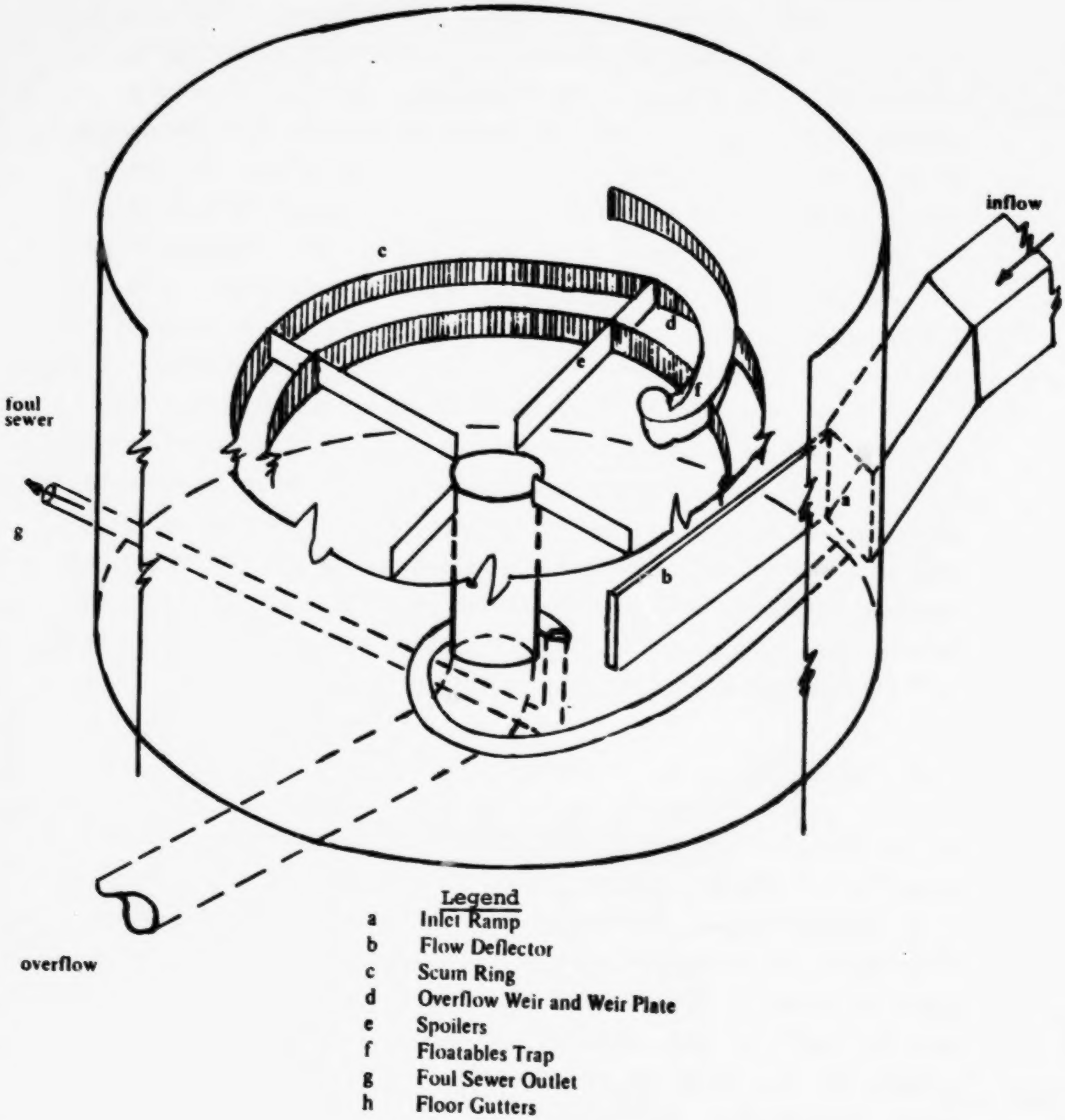

inflow
c
d
e
f
foul sewer
g
a
b
overflow
Legend
a Inlet Ramp
b Flow Deflector
c Scum Ring
d Overflow Weir and Weir Plate
e Spoilers
f Floatables Trap
g Foul Sewer Outlet
h Floor Gutters

Figure 21

Isometric View of Swirl Regulator/Concentrator locations. Consequently, we are deeply involved in the development and demonstration of sophisticated and new flow measuring equipment utilizing the various principles of: hot-film anemometers, concentration of induced foreign matter, ultrasound, and electromagnetics as applied to open channel flow.

Our Program has also contributed towards the development of a prototype monitor capable of instantaneous, in-situ measurement of suspended solids based on the optical principle of light depolarization.

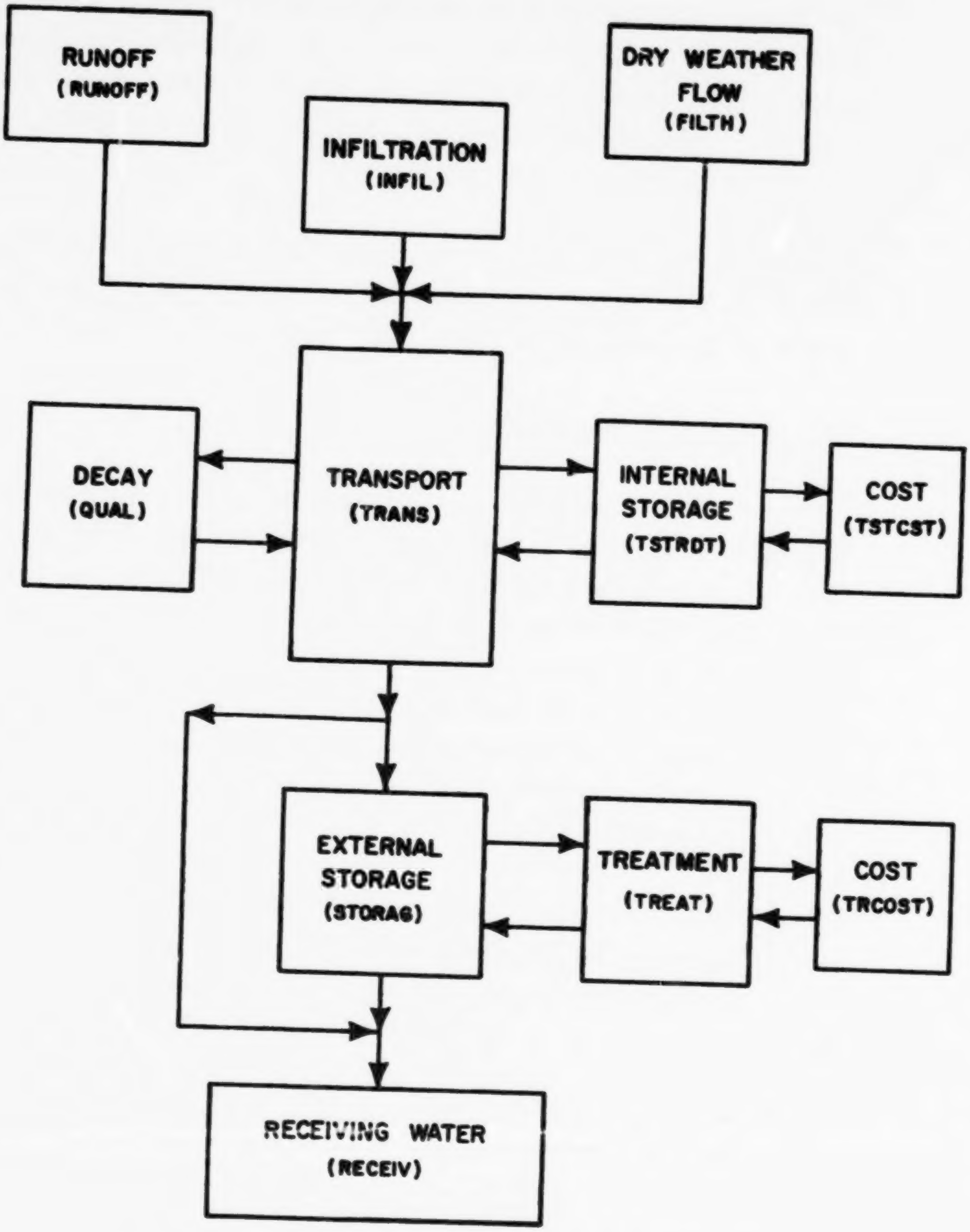
USEPA Stormwater Management Model (SWMM)

The capability to analyse various component flows and pollution loads throughout a sewerage system is one of the keys to better design of control and treatment systems. Due to complexities of the rainfall-runoff-flow phenomena past analyses have been less than adequate, resulting in poor estimates of flow and predicted system responses to a storm. By virtue of previous undertakings, we now have available an operational "descriptive" mathematical model which can overcome former analytical deficiencies. Figure 22 depicts a schematic overview of the model.

We are now in the initial phase of demonstrating the application of this method for "decision-making", that is, its ability to analyse a major combined sewer system to select and to design control and treatment approaches based on cost/effectiveness and to eventually design a computerized means of overall management of the system during storm flows. The model will be fully explained later on today by Dr. Wayne Huber.

PROGRAM PROJECT NEEDS

Looking ahead, the Storm and Combined Sewer Pollution Control Program


RUNOFF
(RUNOFF)
INFILTRATION
(INFIL)
DRY WEATHER FLOW
(FILTH)
DECAY
(QUAL)
TRANSPORT
(TRANS)
INTERNAL STORAGE
(TSTRDT)
COST
(TSTCST)
EXTERNAL STORAGE
(STORAG)
TREATMENT
(TREAT)
COST
(TRCOST)
RECEIVING WATER
(RECEIV)

Note: Subroutine names are shown in parentheses.

Figure 22. OVERVIEW OF MODEL STRUCTURE

needs are vast and numerous. At present, we are directing our efforts to the following:

1. A nation-wide assessment of sewered and non-sewered straight urban runoff impacts, not combined sewage - a consideration which has been stressed by the 1972 Amendments to the FWPC Act.
2. Dual use facilities for wet-weather and dry-weather treatment. Wet-weather facilities built in conjunction with new or existing sanitary plants can demonstrate their synergistic benefit by being utilized to take over during repairs, polishing secondary effluents, or increasing dry-weather treatment capacity during the vast majority of the time, i.e., when it is not raining.
3. Land development making full use of runoff and natural drainage - aesthetically blending into the surrounding environment rather than upsetting it.
4. Wet-weather facilities for treatment of dry-weather creek flow, again making full use of these facilities during otherwise downtime.
5. A stormwater model monitoring/management system for dissemination, updating, and instructions on model application.
6. A functional evaluation of the need for catch basins today - and development of new alternatives.
7. Establishment of uniform techniques for sampling and analysis of storm flow and for determining design volumes and flowrates.
8. Further development of flow measuring devices.
9. Fostering a stormwater survey course at the university graduate level. Storm generated pollution ranks high along with domestic and industrial sources and yet remains unstressed in the schools. With wet-weather control requirements evident, now is the time to encourage universities to cover the concepts of stormwater runoff and combined sewer overflow pollution in proper perspective in their graduate school water pollution

control curriculum.

10. The swirl device applied for grit removal and primary separation of solids from combined sewage, stormwater, erosion runoff, along with the optimization of its sister device, the helical flow regulator/solids separator.

There are also certain major control methods requiring further development. "Upstream" storage or other control processes to decrease the stormwater runoff effect on lower portions of the system is one case in point. Aside from the main objective of controlling storm-generated pollution, upstream control can preclude the need for additional downstream sewer line capacity and associated construction requirements, alleviate shock loadings due to scouring velocities, relieve the often occurring expense of constructing facilities downstream near watercourses in unstable soil with high water table, while offering greater flexibility for control and treatment. An example of this would be the temporary storage or attenuation of stormwater at the building or immediate area through the use of holding tanks, seepage pits (possibly for recharge), roof tops, parks and playgrounds, backyard detention facilities, porous pavement (previously discussed) or neighborhood decentralized stormwater collection sumps including storage facilities under streets. Upstream control systems should automatically regulate discharge from storage to the groundwater, a watercourse, or a sewer system. Plans for reuse of stored water for irrigation, street cleaning, sewer flushing, aesthetic and recreational ponds, potable supply, and other purposes is also encouraged.

Many more ideas and concepts could be added - some may be more significant than those discussed. Submission of ideas, project proposals or grant applications to the USEPA is strongly encouraged.

CONCLUSION

All facts point to a real requirement for treating and controlling stormwater runoff and combined sewer overflows. In view of the tremendous quantities of pollutants bypassed during rainfall from the combined sewer system, it does not seem reasonable to debate whether secondary treatment plants should be designed for 80, 85, or 90% BOD or suspended solids removal, when in fact the small increments gained in this range are completely overshadowed by the bypassing occurring at regulators during wet-weather flow.

The multi-billion dollar treatment plant upgrading and expansion program now going on throughout the country will do much to alleviate pollution of our waters. However, means of mitigating the effects of combined sewers must also be found if we hope to abate the pollution in an optimal manner. Wet-weather standards are already being instituted by the federal government and some states and localities. Recognizing this, our Program will strive to be a prime support for this real world application.

SECTION II

PREVENTION AND CONTROL OF INFILTRATION AND INFLOW

by

Richard H. Sullivan
Assistant Executive Director
American Public Works Association

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The American Public Works Association has had an active program of research in the field of storm water pollution. Its program has investigated such fields as the pollution of storm water, the extent of combined sewer facilities, the design, operation and maintenance of combined sewer overflow regulators and the prevention and correction of excessive infiltration and inflow into sewers. These projects were either conducted under contract with the U. S. Federal Governement or as cost-sharing projects jointly financed by local public agencies and the federal government. My remarks today will be based upon the research findings of our Foundation.

I will briefly review some of the major findings of our report, "Prevention and Control of Infiltration and Inflow". I will also review with you guidelines for the establishment of a survey to determine the nature and extent of infiltration, and some of the factors to be used in making an economic analysis of desirable corrective actions.

In our study of the problems of combined sewer facilities it became evident that infiltration plays a major role in many facilities by either causing more frequent or prolonged overflow events. With the assistance of some 34 local agencies and the Water Quality Office, we undertook a study of the prevention and correction of infiltration. For ease of discussion we decided to consider the "Two I's" of infiltration. The first "I" - infiltration - is in the classic sense, that flow which enters the sewer through pipe and joint defects and manhole covers, etc., and - inflow - is surface water which is deliberately introduced into the system through footing drains, downspouts, area-way drains, and such. Infiltration and inflow both take up capacity within the collection system. However, the two have entirely different characteristics as to time of occurrence, and means of correction and prevention.

If infiltration and inflow exist, why should we be concerned? One of the most common problems associated with excessive infiltration or inflow is backups into basements, flooding of manholes, treatment plant overloads, pavement and sewer failures; all are common problems. Exfiltration may result in pollution of the groundwater table.

When we look at the extent of infiltration, we can conclude that all sewers are combined, it is all a matter of degree. Where even minimal amounts of infiltration and inflow are present, a regulator device of some type will be used on the sanitary sewer system to relieve the excess flow condition. Quite often this is only a leader from a sanitary sewer to a storm sewer, or a hole in the side of a sanitary sewer manhole which, under surcharge conditions, will allow excess flow to enter a creek or stream bed. For such systems to be described as "separate" is ironic, inasmuch as its volume of non-sanitary flow may reach 40 to 1, as contrasted to the strict combined system where this could be 90 to 100 to 1.

Correction of infiltration problems can be categorized under the dual headings of prevention of infiltration and inflow in new systems and the correction of existing conditions.

With regard to new construction; tremendous advances have been made in pipe and joint materials. Contractors and pipe suppliers who worked with the APWA in the preparation of the report were agreed that a construction standard of 200 gallons per inch-mile per day was reasonable and could be met without additional cost to the local agency. In practice we found that consulting engineers had, in effect, an extremely wide array of construction standards which they regularly cite for new construction. There was little agreement as far as to the unit of measure or how the standard would be applied. In this regard I think it is important to remember the effects of a low standard for gallons per inch per mile applied to lengths of 200, 300 and 500 feet. Allowable infiltration may be almost impossible to measure. Specifications using low infiltration rates should spell out how compliance is to be measured. For example: 200 gallons per inch per mile per day allows 4.4 gallons in an 8-inch pipe an hour between manholes 350 feet apart.

The detection of infiltration is a time consuming and generally expensive process. I am not aware of any short cuts to the preparation of a comprehensive survey. Our report contains an outline of a ten-point program as developed by the American Pipe Services Co. of Minneapolis, Minnesota. For purposes of our discussion today I have expanded this to twelve points, and would like to consider these steps briefly with you.

The steps involved in a complete infiltration-inflow analysis include:

1. SET OBJECTIVES: detemine what is the apparent problem, in what condition is the sewer system, is there an adequate

maintenance program, how can sources of infiltration/inflow be determined, and at what cost.

2. IDENTIFY SYSTEM: prepare plot plan of entire system, identifying component drainage systems and key manholes within the system.
3. IDENTIFY SCOPE OF INFILTRATION: make flow measurement, install ground water gauges in manholes, and meter flows at lift pumps.
4. RAINFALL SIMULATION: flood the storm sewer and determine if flow enters the sanitary system - use when infiltration/inflow problems are identified as rain-connected.
5. DETERMINE EXTENT OF SEWER CLEANING NEEDED: a TV camera is not effective unless a sewer line is very clean.
6. MAKE AN ECONOMIC & FEASIBILITY STUDY to determine which portions of the system will be cleaned and physically inspected.
7. CLEAN SEWERS to be inspected.
8. MAKE TELEVISION INSPECTION.
9. DETERMINE EXTENT & LOCATION OF INFLOW.
10. MAKE ECONOMIC ANALYSIS: where should rehabilitation or replacement work be conducted.
11. RESTORE AND REPAIR SYSTEM.
12. ESTABLISH TREATMENT PLANT DESIGN CRITERIA on basis of reduced flows.

One of the important points that must be stressed again and again is that if we are going to look for infiltration we must look when it logically will be present. Thus, the use of groundwater gauges to determine whether or not the individual pipe sections are below the groundwater table is a necessity. Second, the sewer lines must be clean if they are to be inspected. By clean, I mean that a full gauge tool must be passed through the line. This is generally more than the normal cleaning procedure of most agencies. The cleaning procedure will be expensive and time consuming. Therefore, careful analysis must be made as to the capability of the agency to clean sewers and this must be attached with the planned progress of the survey. Cleaning may be a deciding factor in determining how much of a system may be actually investigated. It may be necessary to contract for cleaning.

Properly timed television inspection in a well-cleaned sewer is extremely helpful in analyzing the location and amount of infiltration waters entering the sewer line. Data obtained will include an indication as to locations of many sources of inflow and building sewer infiltration. The latter, building sewer infiltration, is a hard problem to approach, inasmuch as it is very difficult to gain access to that portion of the sewer system. A rough analysis of a community's total sewer system may indicate as much as half of the total sewer system is building sewers. Should the groundwater table be high, and the building sewers under the groundwater table, a substantial portion of the total load may come from this portion of the sewer system. Again, such lines if they are shallow may be an important source of infiltration and inflow during periods of precipitation. One community which experienced severe overloading and basement backups during periods of rainfall found that roof leaders discharged adjacent to a building allowed almost a direct connection of the water from the roof into the building sewer. This community required that roof leaders be discharged five feet from the foundation, and the problem was corrected. In other communities official practice may have allowed foundations drains to be connected to the sanitary sewer. This again leads to a tremendous increase in the flow. In a like manner, sump pumps, if allowed to discharge into the sewer system, quickly cause overloading. Yet another source of inflow water is from manholes. There are many conflicting opinions, however, with regard to using watertight covers on manholes because of the buildup of gas within the system. However, if the manhole is to be located in an area where storm water may enter the system, many communities have gone to watertight covers or have added plugs to the openings to keep storm water out.

Detection of the location of inflow is perhaps the easiest part of the battle. The real test is to attempt to change or correct the conditions within private property. Residents of built-up areas without storm drains in many areas are loathe to have sump pumps discharge onto lawn areas. In fact, in many areas there may not be sufficient lawn area to take the flow. In like manner, foundation drains must have a location and a way of carrying off the flow or there will be backup into the basement. To reduce erosion, roof leaders may be discharged into the sanitary sewer.

The APWA report has recommended that agencies prior to funding reconstruction of paralleling of their interceptor sewer or relief sewer and construction or additional treatment facilities, make a thorough infiltration study to determine the amount of flow which might be eliminated by correction of inflow conditions or improvements of the sewer line to eliminate infiltration.

From a dollars and cents point of view, this seems appropriate. From a standpoint of controlling pollution, we are generally further ahead in eliminating pollution if we clean up the source rather than if we build additional facilities and then have continuing operational cost.

For this reason, in our Manual of Practice, we attempted to develop an outline of an economic analysis in order that the cost of infiltration and inflow waters might be determined and so that an agency could determine how much it could afford to spend for the control of infiltration and inflow. Very few examples were found where such an economic evaluation had been made. While many of the tools that are available at this time are not exact, because of lack of adequate record systems by local agencies, we must have the economic justification of our pollution control activities.

SECTION III

COMBINED SEWER OVERFLOW REGULATOR FACILITIES

by

Richard H. Sullivan
Assistant Executive Director
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There is a broad cross-sectional interest in the proper design and operation of combined sewer overflow regulators. Consulting engineers - general design of facilities; pollution control personnel - monitoring facilities to determine the nature and extent of the pollutional load to receiving waters; industrial representatives - to design and build the actual regulator; and local governmental officials - to bridge between these three groups and to pay for the facilities. Payment is very important inasmuch as for this portion of the pollution control program, federal and state aid is not generally available to assist local government in financing the construction and reconstruction of facilities that will lead to a reduction of this source of the pollutional load. Lack of such aid is somewhat unique and, undoubtedly, is directly responsible for the fact that relatively little work has been accomplished at the local level to implement the types of pollution control programs which have been advocated and demonstrated by the Water Quality Office in the field of storm and combined sewers. Construction grant funds from EPA have been available for only a handful of facilities, where essentially primary treatment will be accomplished.

It is appropriate to consider the "official policy" regarding combined sewers. For many years it appeared that the official policy of the federal government was that combined sewers would be separated. In 1967 the APWA completed its report on the extent of combined sewer facilities with a cost estimate of $48 billion in 1967 dollars to separating systems involving some 36 million persons. It appears that generally the Washington officials are now convinced that separation alone is not the solution, though the word has not necessarily been reached, or been adopted by the regional offices, as we still see results of conferences which will require separation of combined sewers on a wholesale basis. Other federal agencies such as DOT and HUD have also geared their programs to further the separation of combined sewers. This becomes particularly ironic as the extent of stormwater pollution becomes evident and in some areas we begin to talk or require treatment facilities for stormwater. A great deal of rethinking appears warranted at this time before actually establishing a national policy. From the work that the APWA has accomplished, it has been shown that storm waters are polluted whether or not they are carried in separate or combined sewers and that to meet receiving water quality standards, treatment or control facilities may be necessary.

Consulting engineers and local government officials in considering the combined sewer overflow regulator facility problems should begin by defining their needs, particularly in measurable terms. For instance, a general need is to either reduce or eliminate pollution from combined sewer overflows. The need might be based upon a requirement to improve receiving water quality, to improve the value of land adjacent to the overflow, to improve or make possible operation of treatment or control facilities, or to improve operation of the treatment plant. The need, then, must be defined in terms of how much or the extent of actual improvement required. Means must be available to determine whether or not the desired goal has been achieved.

If our desire is to reduce pollution, we should determine whether or not the economical solution is to reduce flow in the combined sewer by a system of surface storage, in-system storage or treatment of the overflow. The type and size of the regulator will vary considerably depending upon the nature of the treatment or control device.

Criteria for the operation of the regulator traditionally has been to limit flow to the interceptor. I would like for you to consider, however, the concept of the Two Q's, control of quality and quantity of the overflow. Regulators can be classified as either static or dynamic. If they are static, they perform in a determined manner, and are unresponsive to changes in control levels in the interceptor or changes in the quality of the sewage. Dynamic regulators, on the other hand, can be designed to be responsive to a variety of flow conditions and flow characteristics. The regulator must be responsive to flow both in the interceptor and collector sewer, the maximum pollutional load should be diverted to the interceptor sewer, there should be no dry weather overflows, there should be low maintenance cost, and a low initial cost is desirable. Operation of the regulator must be responsive to changing conditions. Quality of overflow may be improved by screening, use of secondary motion, or the mode of operation. Choice of the individual regulating device to be used will be influenced by space required, availability of access, outflow conditions, head-loss within the regulator, and exterior power requirements. All must be evaluated and considered.

The major findings and recommendations of the APWA study were:

Efforts should be made by local jurisdictions to consolidate minor overflow points into fewer locations, in which the installation and maintenance of sophisticated regulator devices and controls will be economically and physically justified.

"Total systems" management of sewer system regulator-overflow facilities should be instituted wherever this procedure can be shown to be feasible and economical. This will involve the use of dynamic-type regulator devices and the application of instrumentation and automatic-automation control methods which will be expedited by a reduction in the number of overflow points.

Dynamic-type regulators should be used wherever possible and feasible for "traffic control" of combined sewer flows. This could shunt surcharges of portions of such a system into sections of sewers which are not simultaneously so affected. This approach could be enhanced by the monitoring of precipitation and sewer flows through an adequate network of stations, in communication with a central control point from whence flow routing decisions can emanate.

The type of regulator used should be determined on the basis of its performance and potential reduction in overflow pollutional effects.

Maintenance schedules and budgetary appropriations should be planned on the basis of the specific needs of static, dynamic and instrumented units in service. Each type of regulator should be given the attention it requires to achieve maximum performance.

Regulator facilities should be situated in accessible locations, provided with safe and dependable access facilities, be free of other safety hazards, have adequate space for necessary maintenance work and, when possible, be accessible from locations other than the street or highway right-of-way.

Maintenance crews should be adequately staffed and crews should be provided with all necessary service equipment and tools for their work and for their protection. In-service training should be provided and preventive maintenance schedules should be established. Records of maintenance work must be accurate and complete in order to assess properly the effectiveness of regulator operations and to allocate budget costs for each specific maintenance and operation procedure.

Specifications must require the use of the most servicable corrosion-resistant and moisture and explosion-proof materials in the fabrication and installation of regulator devices and control facilities. The number of movable parts and appurtenances should be reduced as much as possible, commensurate with efforts to provide greater sophistication of regulator facilities.

Where possible, tide gates should be located in adequate chambers. In cases where system control of regulator-overflow networks is provided by automatic-automated means, the proximity of tide gates with regulator chambers will facilitate the tie-in of backwater control with overflow control. State and provincial water pollution control agencies should increase their regulatory control of this source of pollution and provide standard requirements and the engineering personnel necessary for enforcing the control of overflows from combined sewer systems. Further, such agencies must recognize the fact that existing combined sewer systems must be upgraded if pollution levels are to be reduced.

Efforts should be made to design regulators to minimize clogging and consequent pollutional overflows. Where clogging is inevitable, maintenance schedules should be adapted to correct this condition as expeditiously as possible.

As indicated, interest by various states in regulators and overflow pollutional problems vary considerably. Few states have a staff knowledgeable enough to give much guidance to local officials or to even review plans. Many states appear to want to believe that if they do not get too concerned about the problem, it will go away. Many seem to be taking the textbook advice that combined sewers are a thing of the past. Inasmuch as over 30 million people are directly served by combined sewers with some 18,000 overflow points, I doubt that this represents much more than wishful thinking.

At the close of the research project the APWA developed a Manual of Practice. There is a great deal of heretofore unpublished work in it, which represents good practice in the field. Certainly you and the public agency which you serve should review the Manual for information regarding requirements for the design, operation and maintenance of facilities, as well as a description of some of the newer types of regulating devices. Many of you will have a very difficult time convincing an agency that they should pay more than the $2,000 to $4,000 cost of a static regulator device. However, if pollution is to be reduced, time and money spent on the design and construction of adequate regulator facilities will do much to enhance the local program.

SECTION IV

PRESSURE SEWERS

by

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Introduction

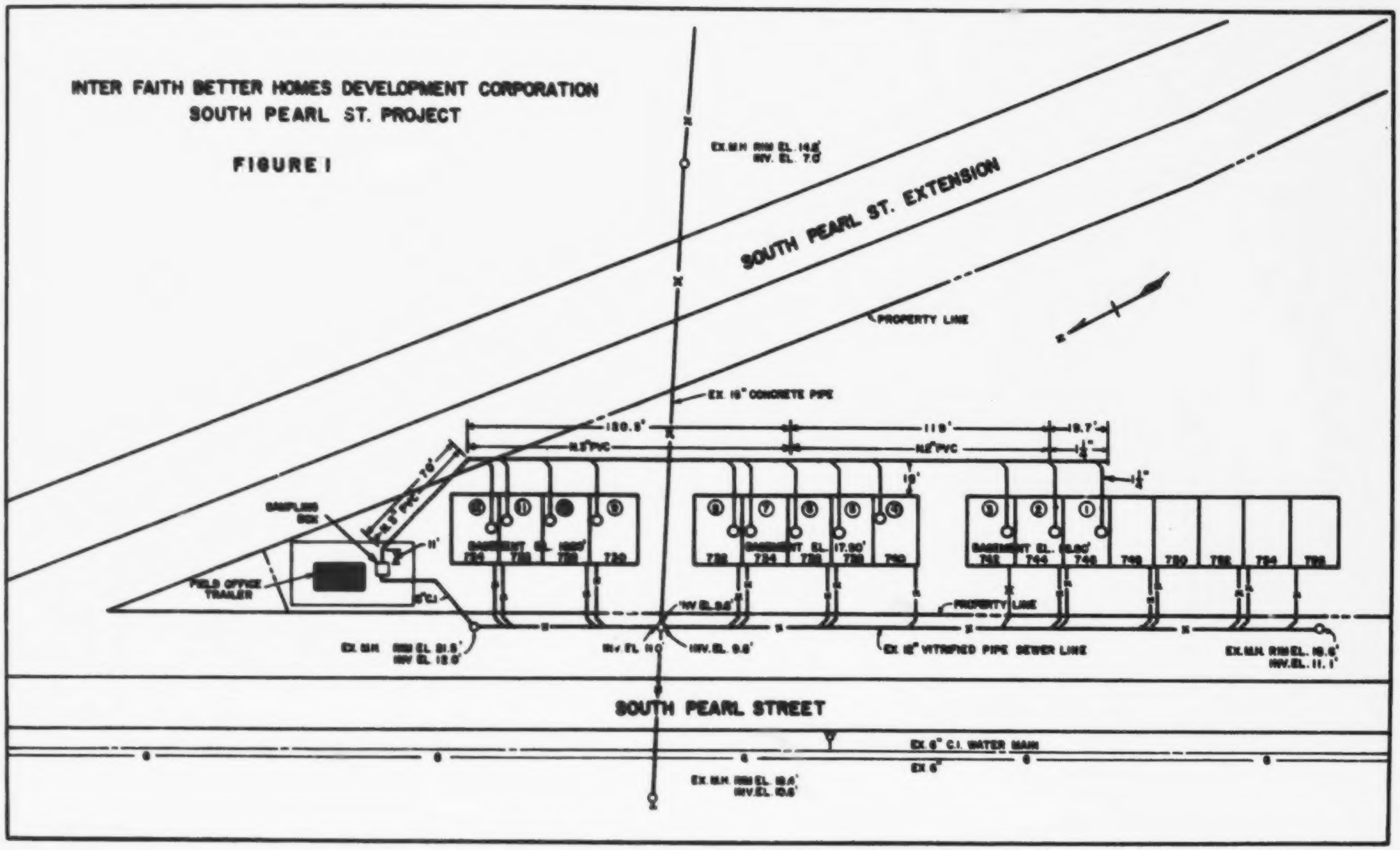
The pressure sewer concept has been around for a number of years. When referring to pressure sewers, we are dealing with a wastewater collection system that utilizes a newly developed Grinder Pump Unit and small diameter plastic or metallic piping systems. It is by no means intended to replace gravity sewers but only to supplement the wastewater collection system.

With financial assistance from both the State and Federal governments, a 13 month study was completed in Albany, New York for the purpose of evaluating the functional specifications of the GP Units and to gain first hand operating experience on the mechanical performance, use pattern, operating cost, maintenance requirements, etc. on these units. The final report is available from the U. S. Government Printing Office(1). A full description of the installations, the monitoring equipment, the piping system, etc. was published previously(2).

Therefore, it is not necessary to go into a detailed description of the installation, with the exception of stating that the pressure sewer system was very simple in design. The wastewater was diverted to the Grinder Pump Unit's tank from which point it was discharged by means of a 1¼" plastic pipe pressure lateral to an outside 1¼" to 3" plastic pressure main. The pressure main at a 4 foot depth received the macerated wastewater from all 12 houses and simply discharged it into a gravity system within the city of Albany (Figure 1).

Grinder-Pump Units

The GP Unit consists of the following mechanical components (Figure 2): (a) Grinder, placed in an inverted position and operating at 1725 ppm with the capability of handling foreign objects without jamming; (b) Pump, positive displacement, progressing cavity type with an almost vertical H-Q curve and proven



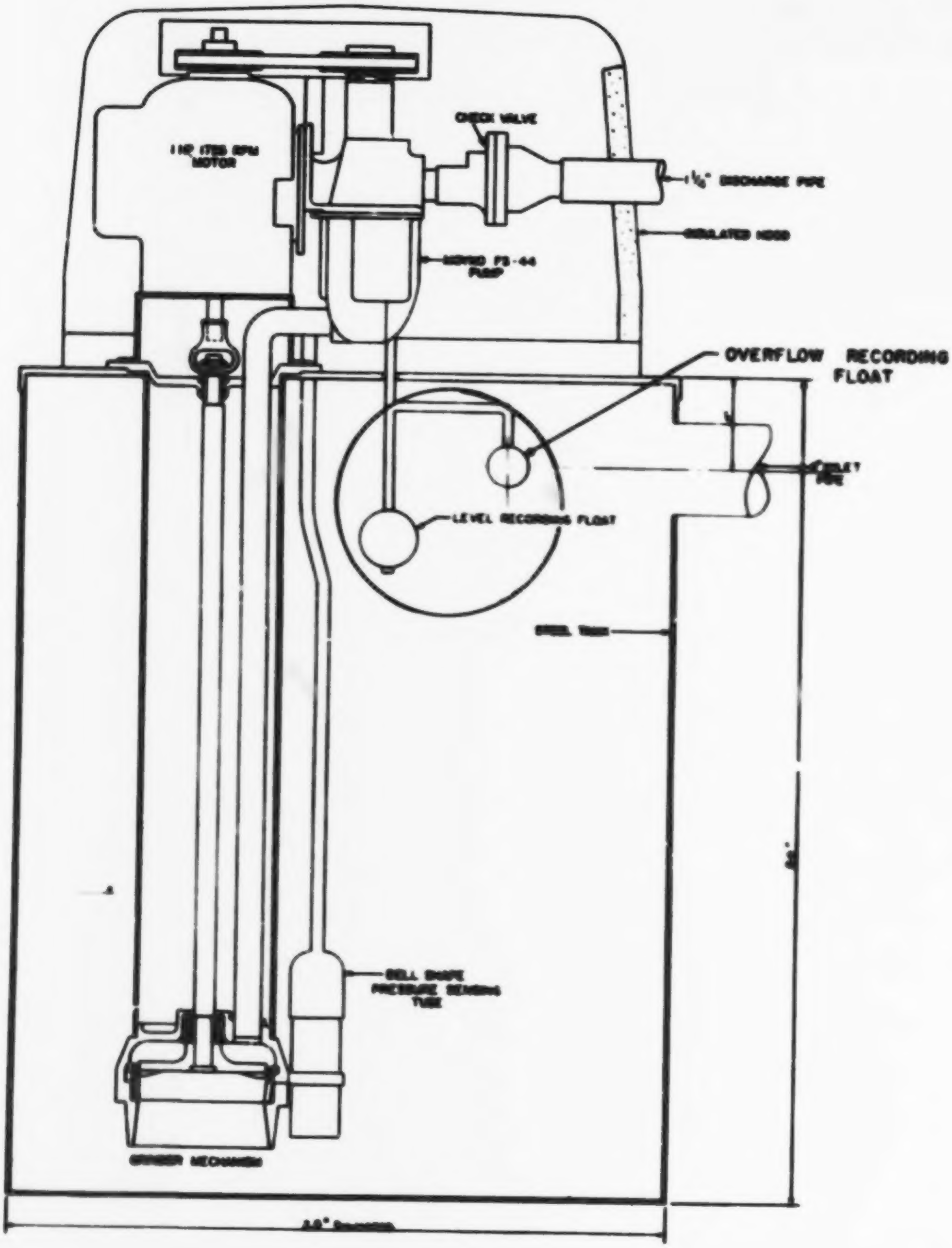
INTER FAITH BETTER HOMES DEVELOPMENT CORPORATION
SOUTH PEARL ST. PROJECT

FIGURE I

EX. M.H. RIM EL. 14.8'
INV. EL. 7.0'
SOUTH PEARL ST. EXTENSION
PROPERTY LINE
EX. 15" CONCRETE PIPE
130.9'
119'
19.7'
11.5" PVC
11.2" PVC
1 1/2"
7.0'
11.5" PVC
SAMPLING BOX
11'
FIELD OFFICE TRAILER
6" C.I.
BASEMENT EL. 18.00'
BASEMENT EL. 17.90'
BASEMENT EL. 18.90'
732
734
736
738
740
742
744
746
748
750
752
754
756
10'
INV. EL. 9.6'
INV. EL. 11.0'
INV. EL. 9.8'
PROPERTY LINE
EX. 12" VITRIFIED PIPE SEWER LINE
EX. M.H. RIM EL. 21.5'
INV. EL. 12.0'
EX. M.H. RIM EL. 19.6'
INV. EL. 11.1'
SOUTH PEARL STREET
EX. 6" C.I. WATER MAIN
EX. 6"
EX. M.H. RIM EL. 19.4'
INV. EL. 10.6'

FIGURE 2

CROSS SECTION OF GP WITH LOCATION OF LEVEL AND OVERFLOW RECORDING FLOATS


1 HP 1725 RPM MOTOR
CHECK VALVE
1 1/4" DISCHARGE PIPE
INSULATED HOOD
MOYNO FS-44 PUMP
OVERFLOW RECORDING FLOAT
INLET PIPE
LEVEL RECORDING FLOAT
STEEL TANK
BELL SHAPE PRESSURE SENSING TUBE
GRINDER MECHANISM
30" Diameter

solids handling ability; (c) Motor, 1.0 horsepower, operating at 1725 RPM, capacitor start, high torque, squirrel cage induction motor with a built-in thermal overload protector; (d) Check Valve, swing check type with passageways smooth and free from roughness and obstructions, and a unique flexible hinge of small section without mechanical pins, rivets, screws, etc; (e) Controls, an inverted diving bell system to turn the motor on and off.

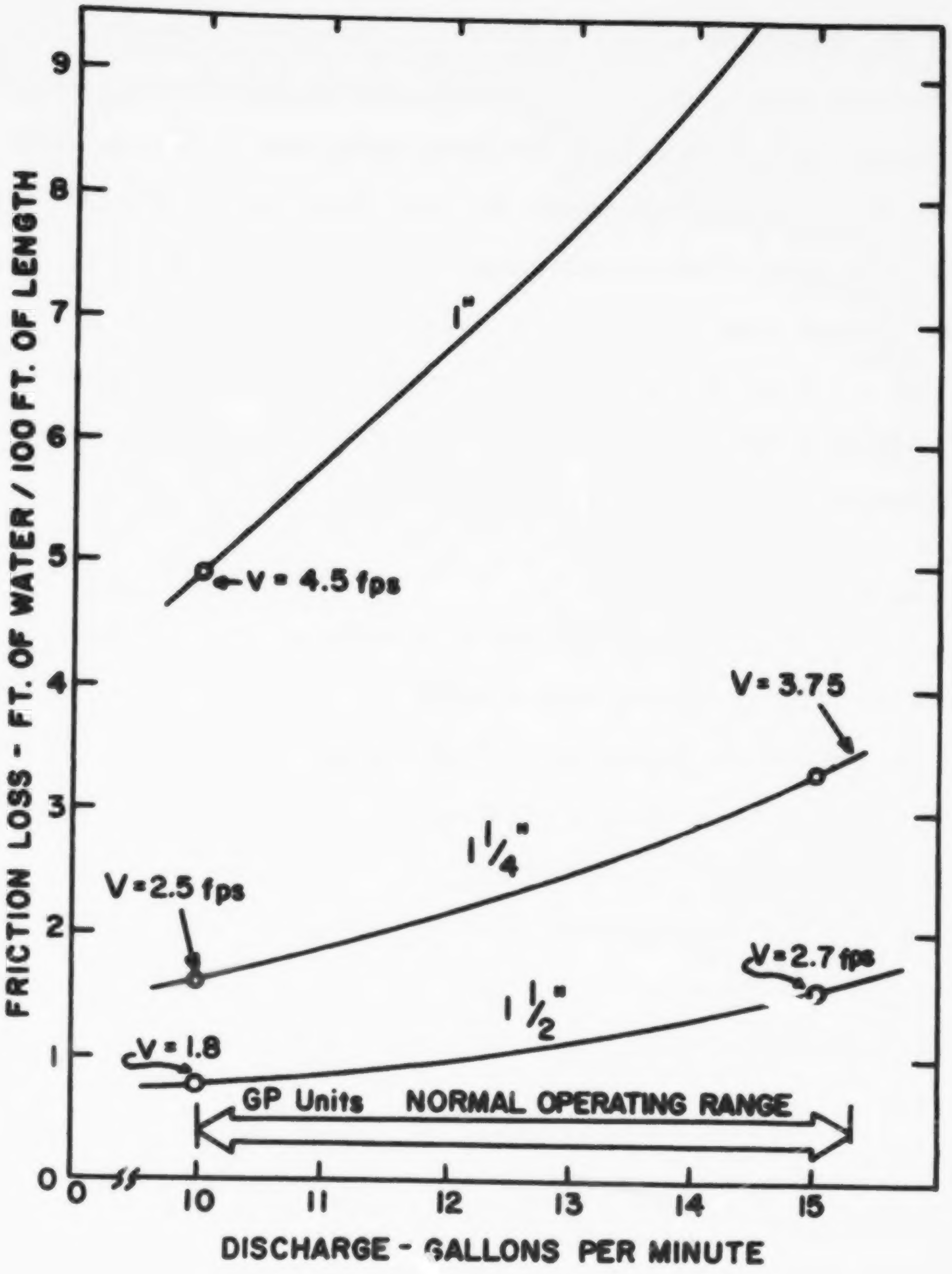
A 1¼ inch discharge pipe was selected as the optimum size(3) capable of not only handling the macerated wastewater without clogging but also minimizing the frictional head losses (Figure 3).

Results

Thirty nine out of the 44 recorded malfunctions were contributed by the Prototype GP Units. Nine of these Prototype units were replaced by Modified GP Units (Figure 4) after only 6 months because of the large number of malfunctions. The newer units performed satisfactorily for the remaining of the project. Loss of prime by pump and grease clogging of the 1" opening within the bell-shaped pressure sensing tube was the major cause of the malfunctions experienced by the Prototype Units. Corrective modifications were incorporated in the manufacturing of the modified GP Units with considerable improvements in the daily operation.

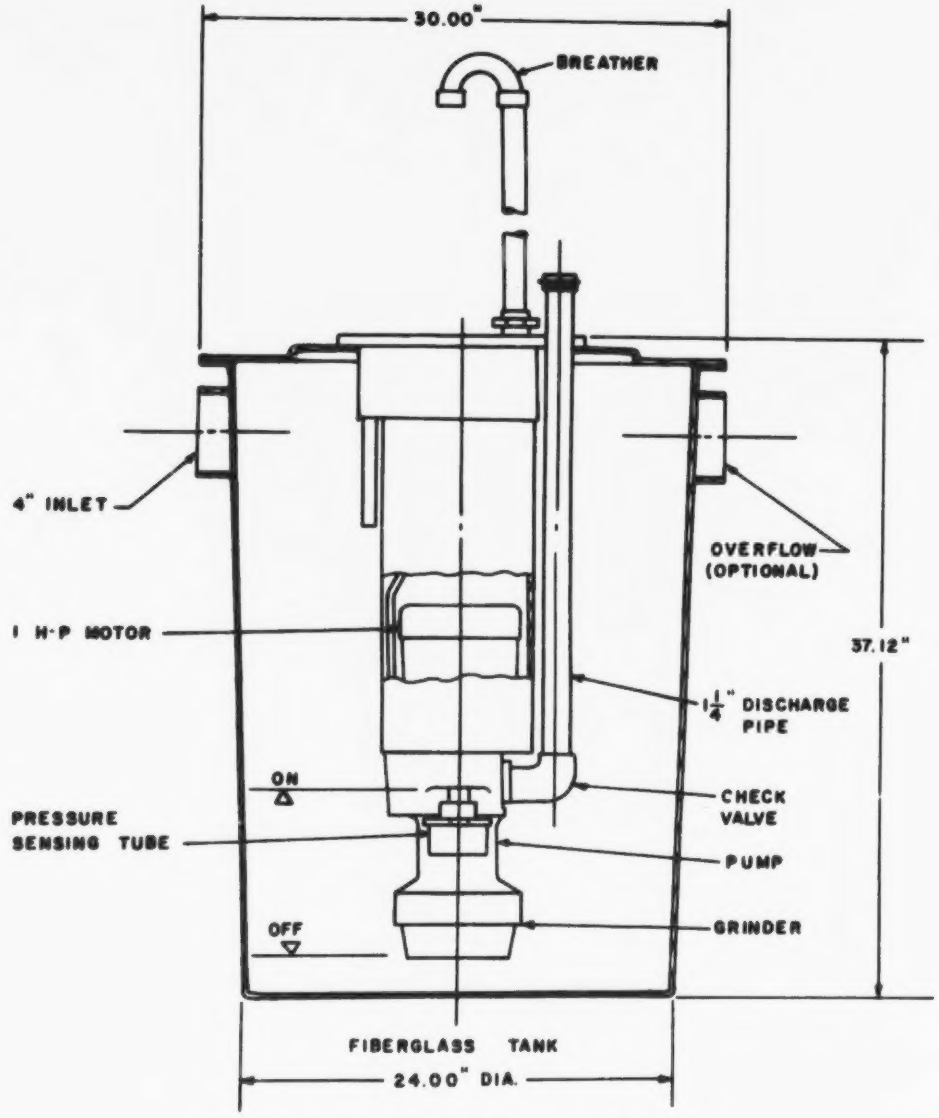
One of the primary interests of this project was to extensively test the reliability of the mechanical components in an actual field installation. Pre-installation testing and post-installation testing (Table 1) was performed in order to determine marked deterioration if any, in the physical structure and performance of the GP Unit's components.

In addition to the 6282 operations which occurred during the so-called



FRICTION LOSS vs DISCHARGE
FOR THREE SIZES OF
POLYETHYENE PIPE

FIGURE 3


30.00"
BREATHER
4" INLET
OVERFLOW (OPTIONAL)
1 H-P MOTOR
37.12"
1¼" DISCHARGE PIPE
ON
CHECK VALVE
PRESSURE SENSING TUBE
PUMP
OFF
GRINDER
FIBERGLASS TANK
24.00" DIA.

CROSS-SECTIONAL VIEW OF MODIFIED GP UNIT

FIGURE 4

Table 5 I

SUMMARY OF OBSERVATIONS AND TESTS OF GRINDER PUMP UNITS AT CONCLUSION OF DEMONSTRATION

Item	Unit Numbers 1	2	3	4	5	6	7	8	9	10	11	12
Operations Counter Reading	11471	5211	11656	7214	5337	1707	6550	7057	9131	8694	3897	8225
kW Hours	154	78	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
Exam Controls for Moisture	OK	OK	OK	OK	OK	NA	NA	OK - Grease in Sensor	OK	NA	NA	OK
Timer Operations (Seconds)												
"On" Delay	35.0/36.3	35.7/25.4	33.9/34.8	32.6/33.0	42.1/41.4	NA	29.1/28.9	32.7/37.2	38.5/38.7	33.7/25.0	26.5/27.8	39.1/39.2
"On" Time	54.2/53.6	59.9/59.5	54.9/53.2	51.8/50.4	54.5/51.0	NA	50.6/52.5	71.1/86.5	50.0/49.3	35.7/40.0	31.4/29.3	48.7/48.3
"Off" Delay	30.5/29.6	26.0/25.5	27.2/26.5	26.6/25.9	25.4/24.2	NA	29.7/29.5	27.3/29.7	28.0/28.4	10.4/11.7	----/11.8	25.9/24.9
Check Valve Buble Tight?	No	No	No	No	Yes	Yes	No	No	No	NA	NA	No
Noise Level	Quiet	Slightly Noisy (Loose Parts)	Quiet, but not as low as 1,5,9	Slightly louder than average	Quiet	NA	Louder than Production Model	Quiet, but something vibrating	Quieter than washer or furnace fan	NA	Very noisy - (Bearing Problem)	Very quiet
Level Control Depth in inches												
"ON"	12½	13	13	13-1/8	12	NA	14-1/8	15¼	13	10½	9-3/4	12-1/2
"OFF"	5½	7	3½	7	5½	NA	7-½	4	5	4-3/8	5	5-1/8
Pump Performance GPM/Watts @												
15 psig	14.8/900	14.7/875	14.8/850	14.8/850	14.8/940	14.8/925	14.2/860	14.8/900	15.2/875	15.4/900	14.8/860	14.8/880
25	13.8/1025	13.6/1000	13.6/1000	13.8/925	13.7/1050	13.8/1050	13.8/940	13.8/1025	14.0/1025	14.5/1000	14.0/960	NA
35	12.4/1125	12.2/1125	11.8/1075	12.5/1100	12.4/1100	12.2/1200	12.0/1060	12.4/1150	12.8/1160	13.4/1100	13.2/1060	12.3/1150

NA = Not Available, or not checked

de-bugging period, a total of 73,458 GP Units operations were recorded during the remainder of the demonstration project (Table 2).

Even though the operating cycle varied greatly for the prototype units, the modified units operated on a cycle between 57 and 74 seconds (Figure 5), with the average operating time of 11.5 minutes to 27.5 minutes per day. Furthermore, based on the occupancy rate of 75 persons for the 12 town houses, a value of 2.6 operating cycles per capita per day was calculated for this particular single family residential development.

The documentation of the operating cost was of prime interest, since it was essential to verify the theoretical cost value of $2.12/year for a family of 5(3,4). Two watt-hour meters were installed to register only the total power consumption of two individual GP Units. Based on the monthly operating time, proportional monthly power consumption values of 10.2 and 5.3 KW were calculated. Applying an average incremental power consumption rate of 2.3¢ per kilowatt hour (KWH), the monthly operational cost for Unit No. 1 amounted to $0.24 and $0.12 for Unit No. 2 (Figure 5), which is equivalent to $1.18 for a family of 3,up to $3.50 for a family of 9.

The GP Unit's usage varied greatly from day to day for any given unit. An even greater variation was documented when comparing weekday versus weekend usage. This is graphically illustrated in Figure 6 for two given units. The total weekend daily usage exceeded the weekday total daily usage by 50-60 operations (an increase of 35% over the weekday total).

As an indication of the improved performance record of Modified Units versus the Prototype Units, a value, known as the "down-time", was computed for each of the GP Units. The "down-time" value is based on the amount of time a unit was non-operational over the total amount of time of possible operation.

TABLE 2 - TOTAL NUMBER OF OPERATIONS

Unit No. Mo Yr	1	2	3	4	5	6	7	8	9	10	11	12	Total per Month
Oct. 70	1097	313	500	383	353	-	393	295	552	332	243	-	4,461
Nov. 70	753	283	509	342	369	-	372	347	575	303	242	-	4,095
Dec. 70	578	295	588	253	380	243	307	350	455	306	201	711	4,667
Jan. 71	993	279	925	456	381	214	359	338	466	250	191	777	5,629
Feb. 71	640	273	818	333	281	198	240	249	422	593	109	601	4,756
Mar. 71	523	264	972	599	299	290	282	701	546	592	124	933	6,125
Apr. 71	399	293	502	394	365	382	383	418	422	551	274	483	4,866
May 71	690	512	524	420	211	209	499	447	525	666	358	604	5,665
June 71	1154	444	493	313	401	-	440	571	577	719	284	600	5,996
July 71	993	483	539	288	295	-	559	828	530	686	370	666	6,237
Aug. 71	886	420	716	599	329	-	487	845	607	775	168	607	6,439
Sept 71	568	366	840	775	238	-	534	735	851	512	226	658	6,303
Oct. 71	950	523	842	761	428	-	814	516	840	500	-	602	6,776
Nov. 71	209	100	166	133	128	-	179	78	168	122	-	160	1,443
Total per Unit	10,433	4848	8934	6049	4458	1535	5848	6718	7536	6906	2790	7402	73,458 Total

Average Operations = 2.6 per capita per day

FIGURE 5

SUMMARY OF OPERATIONAL DATA FOR MODIFIED AND PROTOTYPE GP UNITS

	1*	2*	3*	4*	5*	6°°	7°	8*	9*	10°	11°	12*
OPERATIONS PER DAY	28	15	21	17	10	10	15	22	22	18	8	21
LENGTH OF OPERATING CYCLE (SEC.)	59	71	74	59	69	65	55	57	67	39	40	68
TOTAL OPERATING TIME PER DAY (MIN.)	27.5	17.8	25.9	16.7	11.5	10.8	13.7	20.9	24.6	11.7	5.3	23.8
POWER CONSUMPTION COST PER MONTH ($)	0.24	0.12	0.27	0.17	0.12	0.12	0.14	0.21	0.25	0.12	0.10	0.24

* Modified GP Units

° Prototype GP Units

°° Unit #6 became vacant after May 18,1971.
Values are based on Prototype GP Operations

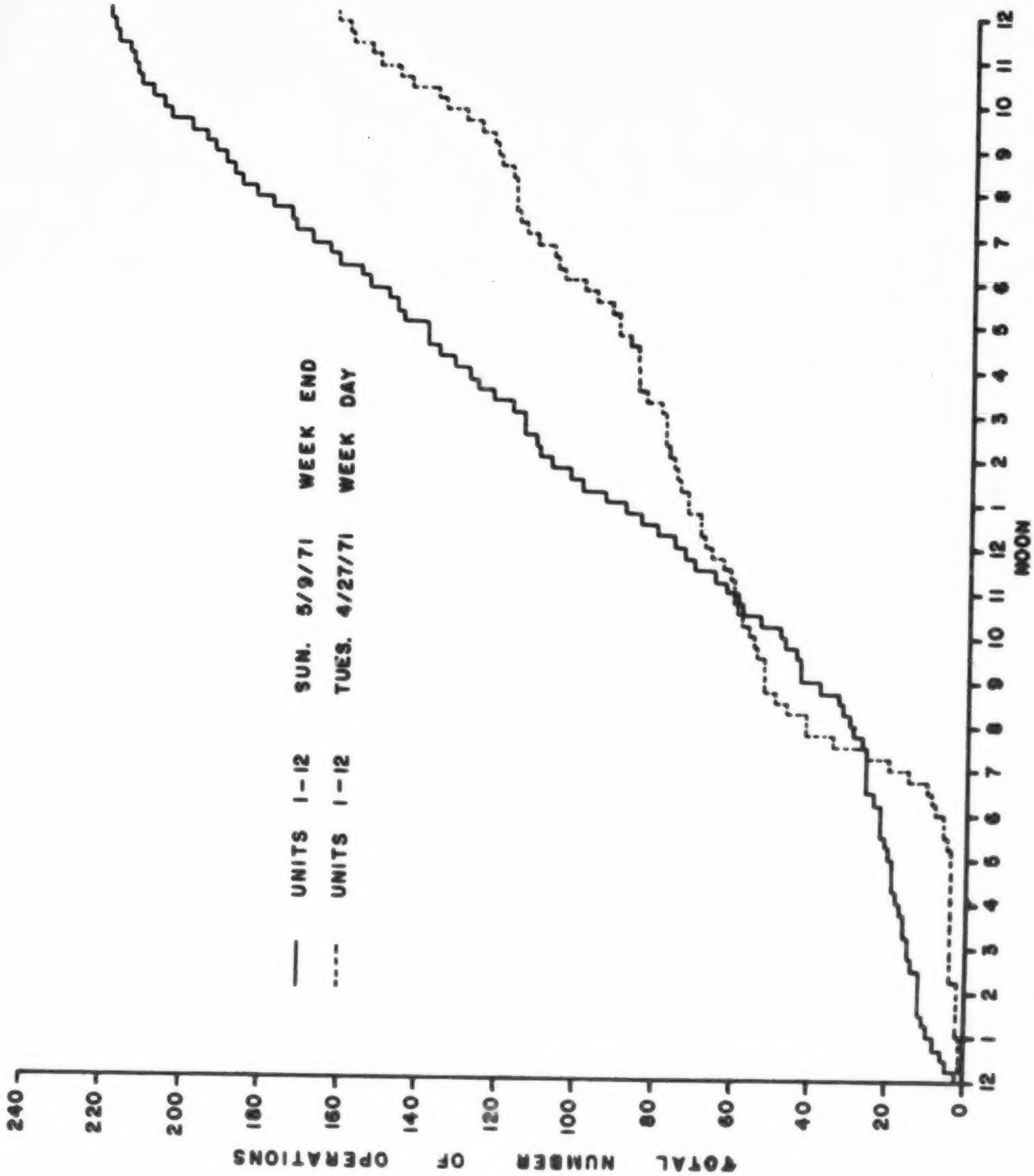


FIGURE 6 TOTAL NUMBER OF OPERATIONS VS TIME OF DAY

The Prototype GP Units produced a "down-time" of 2.69% for the first six months in comparison to only 0.27% for the Modified Units over the last 7½ month period.

Discussion

The pressure sewer system pipe sizing was based on the ASCE minimum scouring velocity criteria of $V_s = \frac{\sqrt{d}}{2}$(5,6) and on certain engineering assumptions regarding the estimated wastewater flows from the 12 GP Units.

It must be understood that the flows in the different portions of the pressure main were based strictly on an engineering estimate. There was no data available on the frequency of GP operations for a multiple units system. It was possible to predict the peak usage hours of the GP Units, but since the operating cycle per GP Unit is very small, 57 secs. to 74 secs., it was almost impossible to predict the number of units working simultaneously during this peak period. It was, therefore, assumed that a maximum flow of 90 gpm would flush regularly that portion of the pressure main serving all 12 GP Units. It must be understood that the hydraulic characteristics of the pressure sewer system is dependent greatly on the varying wastewater flows within that system.

Information on simultaneous occurrences was an essential phase of the project. This type of data is critical for the design of future pressure sewer systems. The maximum anticipated flows will dictate the size of pipe within the pressure system. At the same time, the hydraulic gradient will reach its peak slope. The engineer, therefore, must design a system optimizing the sizes and scouring velocities and be certain that the upper recommended working pressure of the GP Unit is not exceeded.

During the last ten (10) months of the demonstration project, during which time the 12 channel event recorder was in operation, a total of 58,823 operations were recorded, which represent approximately 191 operations per day. Therefore, in order to obtain a picture of the minimum and maximum flows within the pressure system, the above mentioned data indicated that (a) on the average, 2 GP units ran simultaneously 20 times per day (b) 3 GP units operated simultaneously slightly more than once per day, and (c) 4 GP units ran simultaneously on the average of once every 14 days.

Also, by using all the automatically recorded data, total wastewater flows were calculated, which ranged between 95 and 100 percent of the actual water consumption (Figure 7).

The close relationship between the water and calculated wastewater flow is a highly reliable indicator of the corresponding wastewater discharges. Also, winter water flow records can be used to estimate accurately expected wastewater flows.

Pressure gages were installed in each basement so that the maximum and minimum pressures occurring during any fifteen minute period might be recorded. These pressure readings were indicative of the varying hydraulic gradient line for each of the twelve GP units (Figure 8). The computerized data indicate that pressures in excess of 30 psi were reached by a few GP Units.

Once the demonstration phase of the project was completed, portions of the pressure main and the 1¼ in. pressure laterals were carefully excavated and removed. Grease accumulation within most sections was evident. Reductions of up to 40% occurred in the pressure main.

The system was simply overdesigned. Where flows were expected to reach 90 gpm regularly, flows of only 45 gpm were recorded (Figure 9). Therefore, instead of a 3" pressure main, a 2" main would have been sufficient for the 12 town houses.

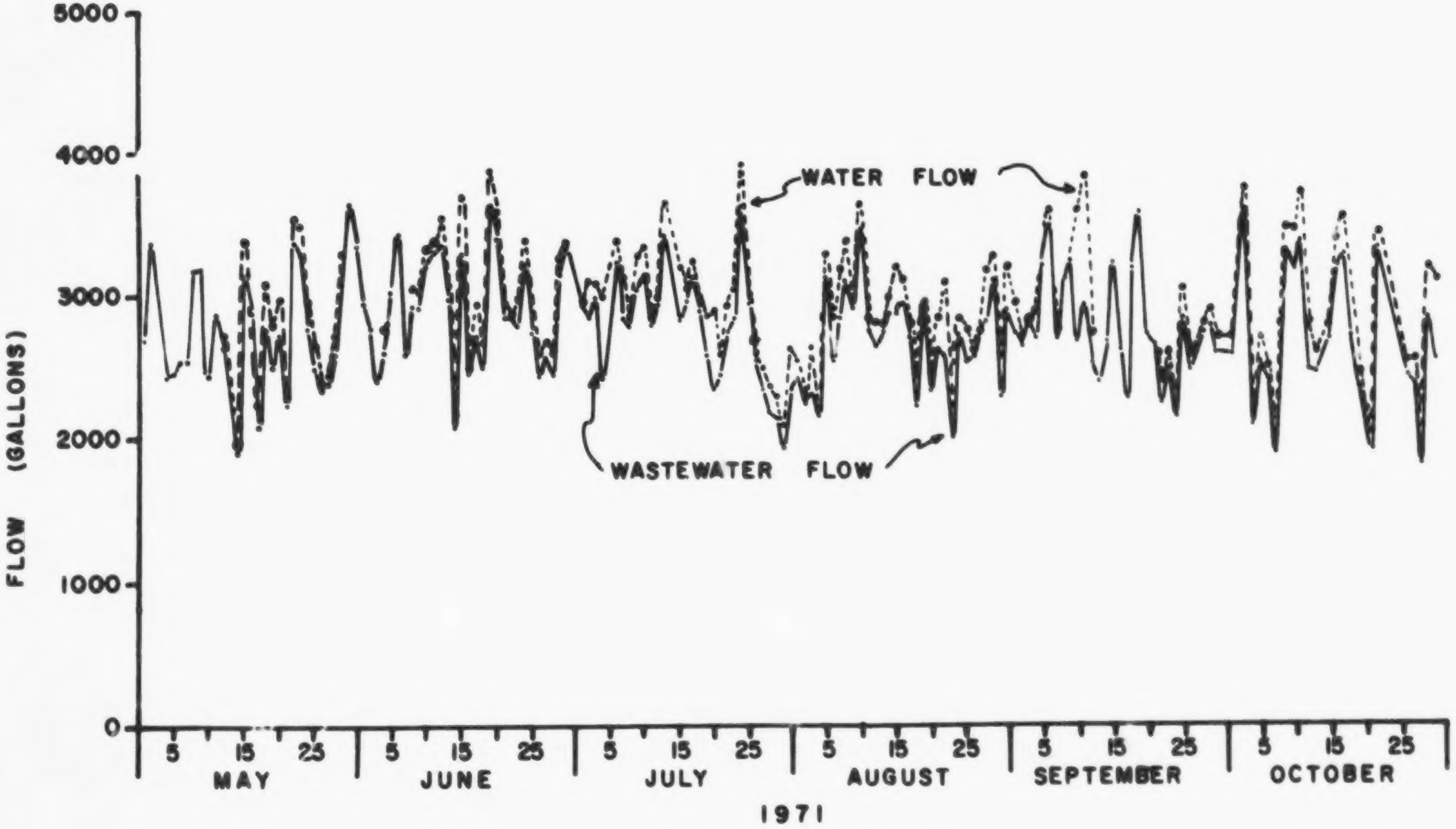


FIGURE 7 WASTEWATER AND WATER FLOWS

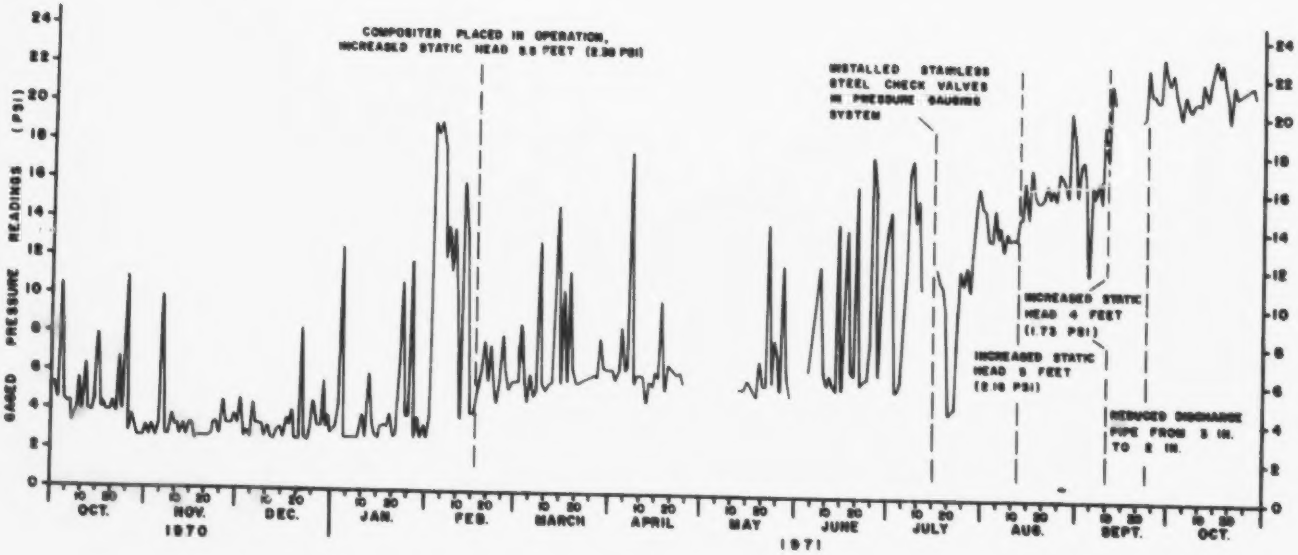
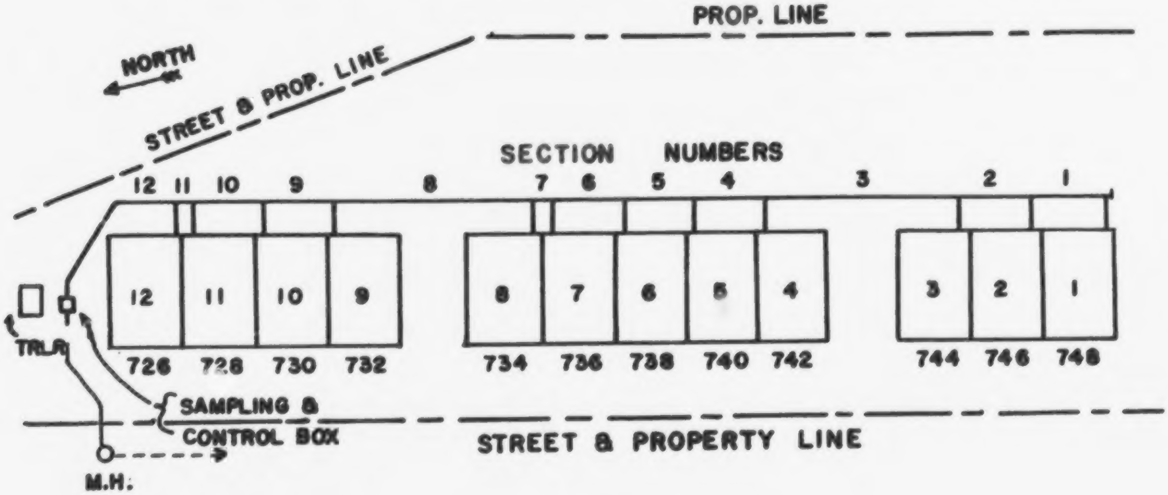


FIGURE 8

GAGED PRESSURE READINGS FOR UNIT NO. 2

FIGURE 9

ASSUMED AND ACTUAL FLOWS FOR THE PRESSURE MAIN


PROP. LINE
NORTH
STREET & PROP. LINE
SECTION NUMBERS
12 11 10 9 8 7 6 5 4 3 2 1
12
11
10
9
8
7
6
5
4
3
2
1
726 728 730 732 734 736 738 740 742 744 746 748
TRLR
SAMPLING & CONTROL BOX
STREET & PROPERTY LINE
M.H.

SECTION NUMBER	PVC-DWV PIPE SIZE	LENGTH OF SECTION (FT)	ASSUMED MAX. FLOW (GPM)	MAX. FLOW RECORDED (GPM)	ASSUMED MIN. FLOW IN 24 hrs (GPM)	DAILY FLOWS (GPM)		
						MAXIMUM	AVERAGE	MINIMUM
1	1.25"	19.7	15	15	15	15	15	15
2	2.0"	20.0	30	30	↓	30	30	↓
3	↓	59.2	45	45	↓	↓	↓	↓
4	↓	19.2	↓	↓	30	45	↓	↓
5	↓	19.5	↓	↓	↓	↓	↓	↓
6	3.0"	19.6	60	↓	↓	↓	↓	↓
7	↓	1.9	↓	60	45	↓	↓	↓
8	↓	58.5	↓	↓	↓	↓	↓	↓
9	↓	17.4	75	↓	↓	↓	↓	↓
10	↓	19.5	↓	↓	60	↓	↓	↓
11	↓	2.7	↓	↓	↓	↓	↓	↓
12	↓	81.0	90	↓	↓	↓	↓	↓

TABLE 3

SUMMARY OF COMPOSITE SAMPLE ANALYTICAL RESULTS

Parameter	Number of Samples	Mean*	Standard Deviation	Minimum Value	Maximum Value
5 Day Biochemical Oxygen Demand	57	330	53	216	504
Chemical Oxygen Damand	56	855	158	570	1450
Soluble Total Organic Carbon	6	140	49	21	225
Total Solids	55	681	87	526	928
Total Volatile Solids	56	476	84	336	706
Total Fixed Solids	56	205	63	57	355
Total Suspended Solids	56	310	77	138	468
Volatile Suspended Solids	56	274	84	78	440
Fixed Suspended Solids	56	36	48	0	268
Total Dissolved Solids	55	372	90	195	637
Volatile Dissolved Solids	55	201	62	22	372
Fixed Dissolved Solids	55	171	58	27	353
Organic Nitrogen**	53	29	12	7	76
Ammonia Nitrogen**	54	51	9	34	68
Nitrate Nitrogen**	38	0.1	-	--	--
Total Phosphate***	63	15.9	6.3	7.2	49.3
Particulate Phosphate***	50	2.8	0.9	0.4	4.2
Filterable Phosphate***	51	13.1	6.5	5.2	47.9
Total Ortho Phosphate***	32	8.7	3.9	1.3	17.9
Methylene Blue-Active Substances****	39	12.4	4.5	4	24
Grease	9	81	12.3	31	140
Settleable Matter ½ Hr.	56	14.5	6.1	4	37
Settleable Matter 1 hr.	56	15.0	6.2	4.5	38
Chlorides	38	52	4	41	61
Hardness	55	65	7.4	46	90
Alkalinity	9	198	8.1	185	209
pH	54	7.8	.3	7.1	8.7

* All values expressed as mg/l except pH

** As nitrogen

*** As phosphorus

**** As linear alkylate sulfonate

There are no existing standards for velocities dealing with the grease accumulation problem, even though velocities in the range of 2 fps to 8 fps have been used by some in designing wastewater pressure conduits. However, for a pressurized sewer system utilizing GP Units, a velocity range of 2 fps to 5 fps is hydraulically and economically preferable.

Extensive chemical analysis were performed (Table 3). The concentration of various pollutants in a pressure sewer system was found to be approximately 100% greater than those found in conventional systems. On a gm/capita/day basis the pressure sewer waste contained approximately 50% less contaminants than reported for conventional domestic sewage. Settleability tests show no significant differences when compared with conventional wastewater.

Therefore, the difference in the strength must be taken into account in designing treatment facilities for a pressure system.

Conclusions

The pressure sewer system, which included the usage of PVC Schedule 40 pipes and PVC-DWV fittings, functioned well for the duration of the demonstration project. Careful considerations must be given to the material used in backfilling pressure main trenches. A good engineering practice is to encase the plastic pipe in sand.

As for the GP Units, the functional specifications have proven to be appropriate. Even though the Prototype Unit exhibited low mechanical reliability, the Modified GP Unit operated to its expectations. Design modifications virtually eliminated all major malfunctions; that is, the 1" opening of the pressure sensing tube was increased to 3" and the pump was relocated so as to be positively primed.

The service record coupled with the "down-time" performance of the Modified Units was impressive, a 0.27% "down-time" value versus a 2.69% "down-time" value for the Prototype GP Units.

Both the pump size and tank volume were more than adequate to handle peak wastewater flows, so that no further design modifications are necessary in this area.

Therefore, in order to summarize the operational performance of the GP Units, a brief review of previously presented facts has been tabulated;

(1) Total Number of GP Operations for the duration of the project - 73,740 operations

(2) Average Operations per capita per day - 2.6

(3) Average Length of operating cycle - 57-74 sec.

(4) Electrical power consumption cost - 34¢/capita/year

In addition, based on the water consumption data, an average wastewater flow of 37 gallon/capita/day was computed. A comparison of the chemical analysis for the pressure sewer project versus the results obtained by others from the conventional gravity systems (Table 4) indicates a much stronger sewage, yet one that contributes 50% less pollutants on the per capita basis. Also, settleability tests indicated no significant difference between the pressure and conventional sewage.

Recommendations

It is recommended that pressure sewer systems be considered as available engineering technology for use where applicable. This recommendation is based on the high mechanical reliability demonstrated by the Modified GP Unit during this demonstration period.

TABLE 4

COMPARISON OF PRESSURE SEWER SYSTEM WASTE WITH CONVENTIONAL WASTE
(From "A Pressure Sewer System Demonstration" by Carcich et al)

Parameter	Conventional Gravity System Concentration mg/l	Conventional Gravity System Loading gm/cap/day	Individual Home Systems Concentration mg/l	Individual Home Systems Loading gm/cap/day	Pressure Sewer System Concentration mg/l	Pressure Sewer System Loading gm/cap/day
5 Day Biochemical Oxygen Demand	180	68	284-542	44-158	330	39
Chemical Oxygen Demand	400	150	540-882	82-205	855	102
Total Solids	700	265	788-1249	113-216	681	81
Total Volatile Solids	350	132	414-659	60-138	476	56
Total Suspended Solids	200	76	293-473	44-106	310	37
Total Dissolved Solids	500	189	-	-	372	44
Settleable Matter(1)	70	-	-	-	15	--
Organic Nitrogen	20	7.5	-	-	29	3.5
Ammonia Nitrogen	11	4.2	48-92	8-16	51	5.9
Total Nitrogen	31	12	61-121	11-20	80	9.4
Total Phosphorus	11	4	15-21	1.9-5.7	16	1.93
Chloride	23	8	-	-	52	6.1
Grease	40	15	33-95	6.1-28	81	10.35
Flow	-	100 gal/cap/day	-	24-78 gal/cap/day		32 gal/cap/day

(1) Expressed as mg/l

The hydraulic design is a very critical facet of any new pressure sewer system. Some grease accumulation can be expected within the pressure pipes. However, critical hydraulic design will limit any excessive grease accumulation and at the same time, will offer the most economical system to the customer. Installation of down-stream clean-outs should be considered as part of good engineering design practice.

Commercially available shut-off valves should be installed in order to isolate the GP Unit from the pressure system during maintenance or repair work. Installation of air relief valves and curb stops should be common practice in as much as the pressurized sewer system is, in many respects, similar to a water distribution system.

Extensive power failures are not very common occurrences in this country. However, since some areas are often hit by power failures due to weather conditions and other uncontrollable factors, it should be noted that GP Unit's tank affords up to 8 hours of wastewater storage, which would be sufficient to accommodate short duration power failures. If historical data from the local power utility dictates a need for an overflow system due to frequent and extensive power failures, then inexpensive cesspools or existing septic tanks can be utilized to receive the wastewater overflows from the GP Units.

At the outset of this demonstration project, emphasis was placed on the fact that the pressure sewer system concept was not meant to replace the conventional wastewater collection system. Rather, the pressurized system is to be used as a supplemental engineering tool for optimizing any wastewater collection system. The economic analysis and sound engineering practice will dictate the extent of the usage of the pressure sewer system instead of the conventional gravity system.

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SECTION V

APPLICATION OF MICROSTRAINING TO COMBINED SEWER OVERFLOW

by

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Combined sewer overflow is a mixture of stormwater and sanitary flow. The special problems of dealing with this flow are due almost exclusively to the stormwater component. Thus, these remarks should apply equally well to overflow of separate storm sewers.

The two components - stormwater and sanitary waste - are somewhat similar in composition. Both contain suspended solids, BOD, and coliform concentrations equal to many times the usual secondary effluent standards. On an annual basis our eleven acre drainage area produces some 9,000,000 gallons of sanitary flow and about 3,000,000 gallons of storm runoff.

The flow rate of stormwater runoff, however, is very high and widely variable. At our site, we have monitored several storms a year where the runoff rate is over 400 times the mean dry weather sanitary flow. It is the flow rate aspect of combined (and separate) sewer overflow that requires a totally different approach when treatment is considered.

Only recently have we become aware of the magnitude of the possible pollutional load from stormwater runoff and have considered treating it. It is not surprising that there is a considerable difference in opinion as to what a stormwater treatment facility should be able to do. The two basic dimensions of a combined sewer (or separate storm sewer) overflow treatment facility are:

(a) The instantaneous flow rate it can handle, and

(b) the amount of each type of pollutant it can remove.

In our studies we have used a flow rate of 2.0 cfs/acre (1.34 mgd/acre) as the required instantaneous capacity of the treatment facility. This runoff rate would require (at a runoff coefficient of 0.4) 4.5 inches per hour rain intensity. At our site we have this intensity sustained for about 15 minutes every 10 years. Analyses of very large drainage areas such as the Boston and Chicago stormwater tunnels where rainfall does not occur over the entire area simultaneously, and where there is tremendous surge volume within the sewer (tunnel), have led to the adoption of a flow rate of 0.2 cfs/acre (0.13 mgd/acre) based on the area of the entire basin. Less understandable is the adoption of low (0.2 - 0.3 cfs/acre) instantaneous design rate for the treatment of combined sewer overflow from small drainage areas of 100 acres or so. Additional experience will permit the selection of realistic design rates for each situation.

It has been suggested that flow equalization basins be included above ground as part of the overflow treatment facility to reduce the peak instantaneous flow rate. Above ground, flow rate equalization basins by themselves may be an attractive scheme of treating overflows, providing space at low cost is available. In this scheme, the peak overflow rate is reduced to a rate where the existing interceptor sewer and sewage plant can handle it as an alternative to an on-site combined sewer treatment facility. Although the annual stormwater volume is some 35% of the sanitary volume, only some 15% additional flow rate capacity would be required.

Flow equalization is most attractive where the subsequent treatment techniques are very expensive on a dollar/cfs peak capacity basis. Flow equalization is essential where the subsequent treatment techniques cannot accept sudden starts and stops or rapid changes in flow rate of several hundred times the dry weather flow variation.

The extent of treatment to be required on combined sewer overflow is at present not standardized. It is not certain what form regulations will take. As will be seen later the familiar "percentage removal" type regulation would be most inappropriate for this problem. Much more work and study must be completed before it can be decided whether it is necessary or consistent with the cost to design overflow treatment facilities for a 25 year return storm or a 5 year return storm.

With current practice, the combined sewer overflow regulator is adjusted to overflow when the rate exceeds perhaps 3-5 times the mean dry weather flow. Thus, the composition of the combined sewer overflow is 1 part sewage to at least 1-1/2 parts of storm runoff. Frequently the composition is over 100 parts of storm runoff to 1 part of sanitary flow. In any event, when significant overflows occur, the composition of the overflow water is determined almost exclusively by the composition of the storm runoff.

The wide range of contaminant levels in the combined sewer overflow reflect the breadth of the range in the storm runoff.

The contaminant level in the combined sewer overflow observed in our site is shown in Table 1.

Table 1

Contaminant		Minimum	Mean	Maximum
Suspended solids	mg/l	15	100	700
BOD_5	mg/l	8	800	3,000
Total coliform cells/100 ml		1,000	1,000,000	3,000,000

Previously we had found (during the fall and winter storms) that, in general, the contaminant concentrations were higher on the bigger storms particularly in the case of the suspended solids. Recently, however, (during spring and summer storms) we found little relation between storm intensity and contaminant levels. The BOD and coliform content of overflow do not seem to have any relation to storm intensity but do seem to have an annual variation. Each drainage area has no doubt a unique combination of features which will influence the character of the stormwater overflows. Our experience, however, has been paralleled by the reported observations of others. They find that sustained higher contaminant concentration levels are as likely if not more likely to occur in large overflows from the bigger storms as from the smaller overflows from less intense storms.

Thus, the treatment design criteria and the regulations must, for the present, assume that maximum overflow contamination concentration will exist at design peak flow rate. More work is needed on this aspect. To attack a given combined sewer overflow situation, the first step is to predict the peak rate-duration and frequency of the actual overflows. With these predictions at hand a decision to treat all storms of less than a certain return frequency must be made more or less arbitrarily. One method of arranging the storm flow data is that used by Dow (2). See Figure 2 from that report. Note that treating about one-third of the peak flow observed over an 8 year study would treat some 98% the total annual flow.

The benefit of flow equalization can be evaluated for the storms to be treated. That is, the relation between equalization basin volume and the reduced peak rate can be ascertained. This work might be extended to, say, 60 minutes, which will be the residence time of some of the actual treatment techniques. We will return to this flow rate consideration after we look at the degree of treatment needed.

There is paucity of information regarding the impact of combined sewer overflow contaminants on the receiving stream. It seems that the pounds of suspended solids discharged per year would be an important criterion.

It is not known how much greater impact these solids would have when they are discharged in slugs of approximately 40-60 hours annual duration. If it is found that the instantaneous rate of solids discharge is significant, the regulations may be phrased in terms of maximum pounds per hour. This is a very complex problem and the methods of considering it have not been developed.

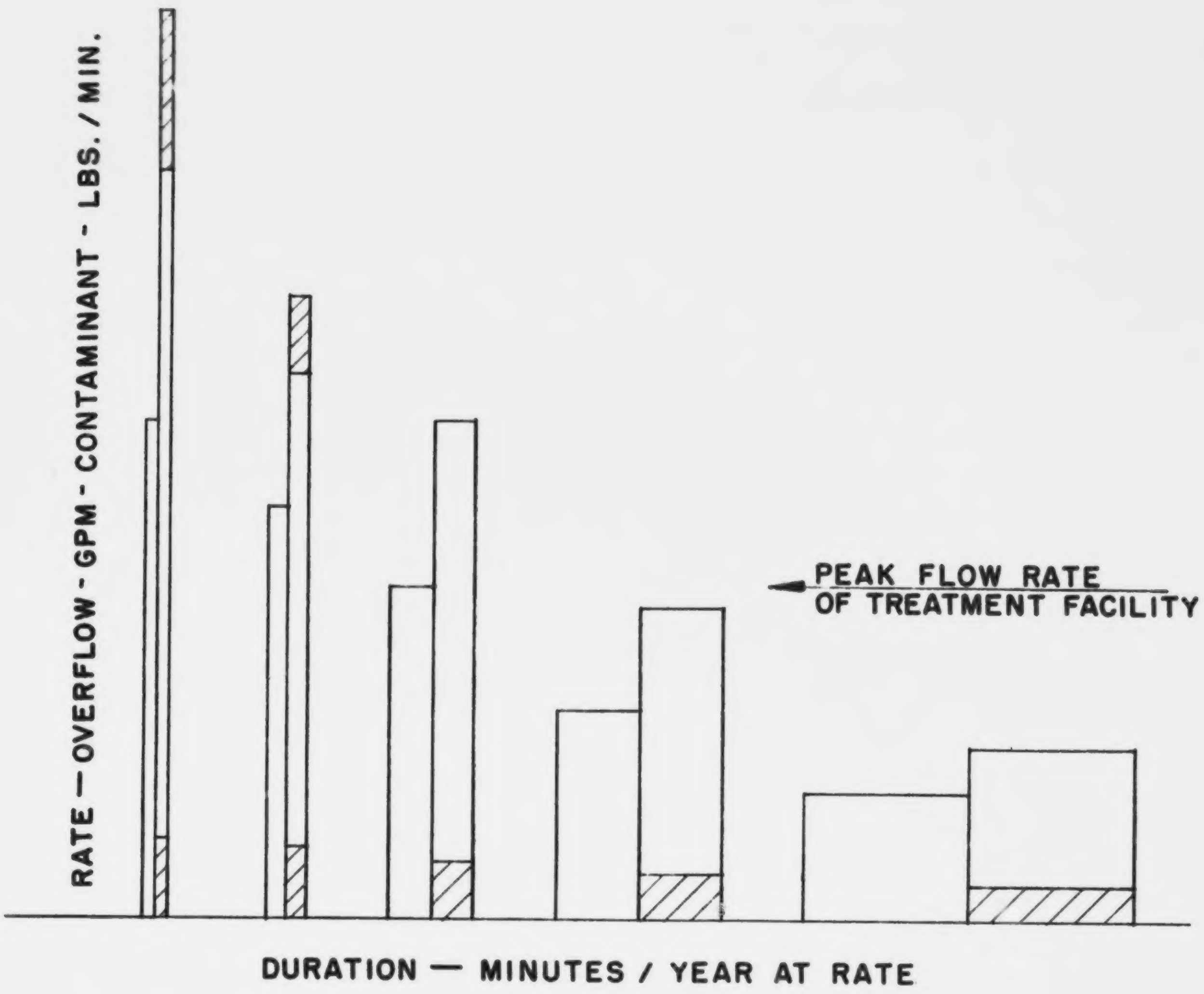


FIGURE 1
DISCHARGE RATE - ANNUAL VOLUME
OF OVERFLOW - CONTAMINANTS

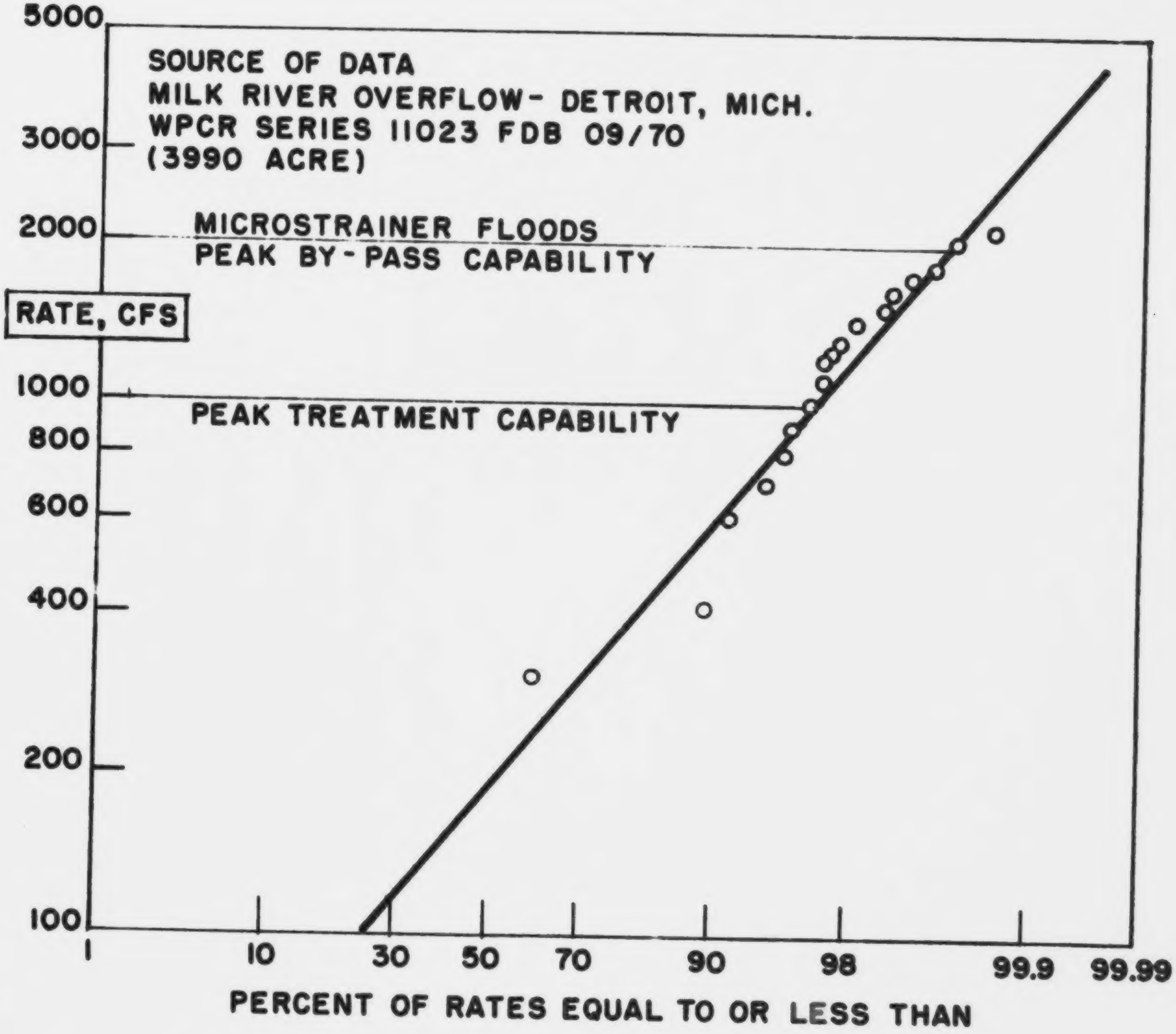


FIGURE 2
DISTRIBUTION OF STORM OVERFLOW RATES (1960-1968)

The potential pollutant load of untreated combined sewer overflow during a big storm is: (Overflow Rate) x (Pollutant Concentration; e.g., S.S.).

The potential load can be reduced by treatment to a lower level, depending upon the design of the treatment facility as follows:

(Overflow Rate-Peak Capacity) x (Pollutant Concentration) plus

(Peak Capacity) x (Pollutant Leakage).

Figure 1 is a preliminary attempt to illustrate this relationship in a stylized manner. The bars represent overflows in increments of magnitude. The height of the bar represents the magnitude of the flow (the left of the pair) and of the instantaneous contaminant flow; e.g., pounds of suspended solids per second. The width of the bar represents the duration of flow of the indicated magnitude in minutes per year. The area of the bars then represent overflow volume per year at indicated rate (left of pair) and the pounds of contaminant per year. The shaded area at the bottom of the solids bar represents the solids leaking through treatment facility and entering the stream. An arbitrarily selected design peak flow rate for a treatment facility is shown. The shaded area on the solids bars representing the biggest storms shows the additional solids entering the stream by direct bypass of the facility.

The amount of the annual contaminant load to the river of the design parameters - peak flow capability of the facility and the leakage through the facility can be seen. Also, the instantaneous rate of contaminant discharge can be seen.

Figure 2 shows another way to consider the overflow rate-annual duration data.

In the previous application section, I have attempted to show the importance of Peak Flow Rate Capability of a combined sewer overflow treatment technique (s). Also I tried to show the importance of Contaminant Level Removal Capability of treatment techniques at design (peak) rate and below design rates.

The announced subject of this paper is a description of the capability of the Microstraining technique in this service.

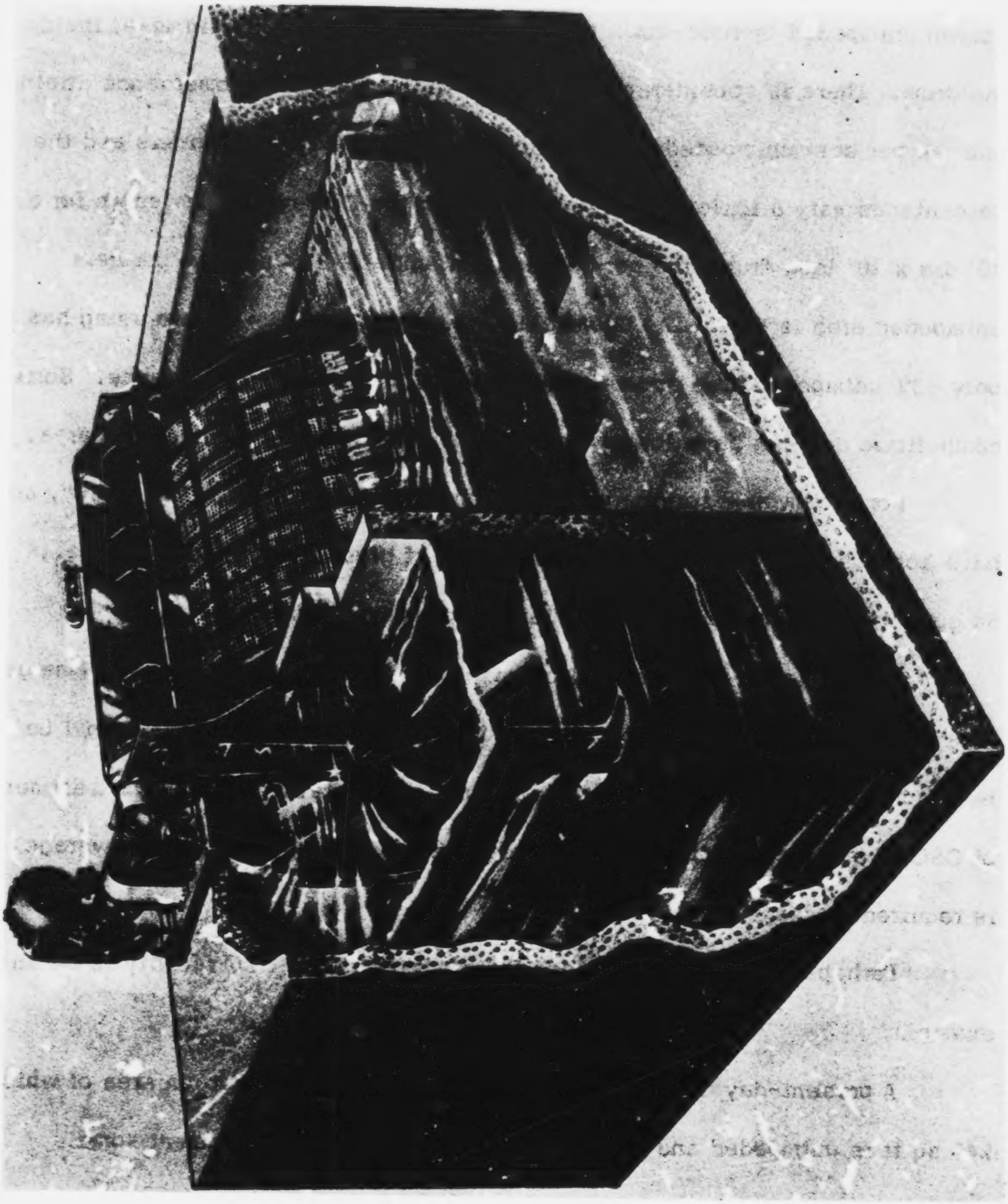
Figure 3 is an isometric drawing of a microstrainer. A microstrainer is a rotating drum fitted with fine screen. For stormwater the screen used is what we call Mark 0, a stainless steel Dutch twill screen with 600 x 125 wires per inch yielding about 23 micron (1-1/2 millions of an inch) apertures.

The stormwater enters the open end of the drum and passes through the screen into the outlet chamber and then to waste. The suspended solids are retained by the screen. As the drum rotates, the screen with a mat of retained solids on the inside is brought up and under a row of backwash jets which wash the solids off into a hopper and thence to disposal. The backwash water requirement is about 1-1/2 gpm per foot of drum length which is a fraction of a per cent of the thruput capability. The solids-rich backwash water stream is small - less than the DWMF - and can easily be sent via the interceptor to the sewage plant, for smaller CSO facilities, or disposed of locally. The backwash water source can be repumped microstrained CSO or preferably city water on small unattended satellite facilities.

The flow of water through the screen is motivated by the difference in level inside the drum over the level outside the drum. In conventional applications of Microstraining this differential is about 6 inches. At this differential

Figure 3

Isometric Drawing of a Microstrainer



the Mark 0 screen will pass only about 6-8 gpm/ft^2 of gross submerged screen area. It might be noted here that the flow capability is not based upon the gross area of the drum but rather upon the open submerged area. That is, that area of screen unimpeded by hold-down straps which lie below the liquid level inside the drum. There is considerable difference in the per cent submergence attained and the per cent unimpeded area in currently available microstrainers and the percentages vary a little from size to size. In the Current Crane design for a 10' dia x 10' long drum, the per cent submergence is 83% and the per cent unimpeded area is 94%. The Glenfield-Crane (older) design we are using has only 83% unimpeded area and was adapted to achieve 83% submergence. Some competitive designs have lower percentage submergence and unimpeded area.

For stormwater service we use much higher differentials, up to 24", and have achieved flow rates of up to 45 gpm/ft^2 of gross submerged area (i.e., 54 gpm/ft^2 of unimpeded submerged area) with very high removals.

The following remarks will be based upon 35 gpm/ft^2 of gross submerged area (42 gpm/ft^2 of unimpeded submerged area). Also, these remarks will be based primarily on the use of a microstrainer as a satellite station for treatment of CSO; i.e., located at the point of overflow so that no additional sewerage is required.

Perhaps the best way to describe a microstrainer CSO facility is by an example.

A present-day Crane 10 x 10 has 314 gross sq ft of screen area of which 245 sq ft is unimpeded and submergible. Such a machine can treat some

10,500 gpm or 23 cu ft/sec of any of the combined sewer overflows we have seen in 16 months of study. Our example will be a facility with two such machines in parallel. (As previously mentioned, the 46 cfs (30 mgd) flow capability of these two machines would be required by a drainage area of from 24 to 240 acres depending on many factors unrelated to the microstrainer.)

Any CSO treatment facility will require a coarse bar screen. The space for and the cost of a travelling bar screen have been included in this example facility. Almost certainly any CSO treatment facility will be sized to treat something less than the peak storm that will occur in the life of the equipment. Thus, a bypass arrangement is required to divert the flow in excess of the peak capacity of the treatment equipment without interfering with the capability of the equipment to treat its peak flow. This consideration may be less important with Microstraining than with other techniques. A microstrainer will flood; i.e., untreated water will overflow the washwater hopper at inlet levels 3" or so above the design level at peak design flow rate. The microstrainer cannot, however, dump previously removed solids into the effluent under excess flow conditions. The space for and the cost of a bypass weir and channel suitable to divert excess flow equal to the design flow have been included in this facility. That is, this facility can accept 92 cfs, treat 46 cfs without hinderance, and bypass the remainder to the receiving stream, or rather to the disinfection chamber, and then to the stream.

The bar screen-microstrainer facility with flumes and chambers for bar screening of 92 cfs, Microstraining of 46 cfs, and bypass of 46 cfs will occupy a ground area of 30 x 40 ft x 10 ft deep. The facility area of 1200 sq ft of ground area is 1/35 acre or about 1/1000 to 1/10,000 of the drainage basin. The liquid volume of the facility is about 9,200 ft^3, or 200 sec residence, at peak flow. The head loss through the facility is about 3 ft during peak flow. While 3 ft is the minimum head required during a storm, ideally there should be 10 ft of head available so that the facility can be drained by gravity after the storm. Otherwise, a small (3 hp) sump pump will be required.

The chamber will be comprised of about 2,500 sq ft of concrete walls and 1,200 sq ft of floor, and to put it below ground will require about 600 yards of excavation.

The microstrainer section should be housed and kept above freezing. The recommended building then would be about 16' x 40' x 18' high. The individual microstrainer units weigh about 13,000 pounds and an I beam craneway should be provided for installation and maintenance. An insulated Butler Building of this size is included in the cost data.

To keep the microscreen in condition to operate when needed it must not be allowed to become dry while soiled. The recommended procedure for combined sewer overflow service then is following a storm to drain the chamber, continue the backwash of the slowly rotating drum using city water as washwater for several hours and then stop the drum and the backwash water.

Also, for sustained dry periods the drum can be rotated slowly for short periods at intervals under backwash jets and the UV lights. The program controls for carrying out this maintenance operation automatically are included in the cost data.

The cost of a complete facility installed, less land and engineering, was estimated to be \$195,000 in 1969 dollars. This investment represents an annual capital charge of about \$19,500/year to be applied to the facility. This annual capital charge is, by far, the major cost for Microstraining (or other techniques) for combined sewer overflow. This cost applied to the drainage area represents about \$80 to \$800 per acre at peak design rating of 0.2 and 2.0 cfs/acre respectively.

The effect of scale on the cost of a facility can be seen in Figure 4.

The utilities required for the two machine facility include about 50 gpm of city water. The electrical power demand is for two 5 hp drum drive motors, a 3 hp sump pump, if required, a 5 hp drive for the automatic bar screen rake, and for lighting and controls - about 25 kilowatt connected load in all. With 50 overflow events a year (we see only 40), and several hundred short, dry weather periods of operation, the running time then will be 280 hours a year so that the annual power consumption will be 7,000 kwh/year or abour \$140/year. Similarily, the city water consumption will be about 14,000 gallons/year most of which is consumed during rainy weather.

The microstrainer is automated. At onset of storm overflow the liquid level in the inlet channel rises and actuates a level switch which starts the microstrainer drum motor, the backwash jets, turns on the UV lights, and the bar screen rake drive.

The microstrainer drum speed controls regulate the speed of the drum in accordance with the difference in liquid level across the screen which is roughly proportional to the flow rate. All of the combined sewer overflow passes through

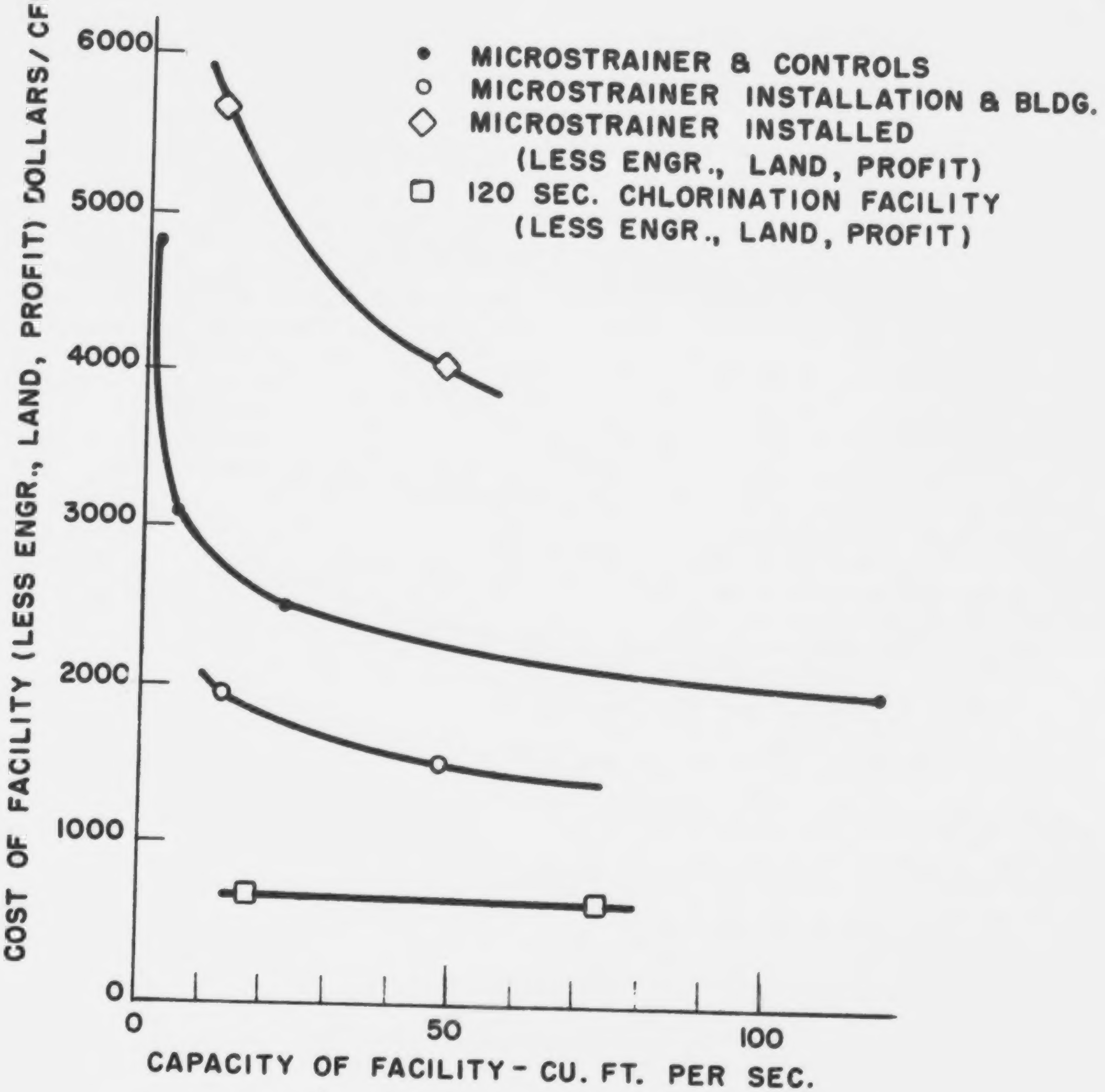

● MICROSTRAINER & CONTROLS
○ MICROSTRAINER INSTALLATION & BLDG.
◇ MICROSTRAINER INSTALLED (LESS ENGR., LAND, PROFIT)
☐ 120 SEC. CHLORINATION FACILITY (LESS ENGR., LAND, PROFIT)
COST OF FACILITY (LESS ENGR., LAND, PROFIT) DOLLARS/CFS
6000
5000
4000
3000
2000
1000
0
0
50
100
CAPACITY OF FACILITY - CU. FT. PER SEC.

FIGURE 4

the drum. If the storm flow should exceed the peak design rate of the machines (i.e., cause a differential in excess of 24") the excess water overflows the bypass weirs and flows directly to the receiving stream or to the disinfection facility and then to the stream. At the end of the storm, the program controls continue the operation of the microstrainer, sump pumps, etc., until the chamber is drained and the screen is clean and then shut them down. The instant readiness and the very low residence volume of the Microstraining technique permits unattended operation with very simple controls. Our equipment ran on all storms under automatic controls. It was unattended during the first part of all storms. No trouble was observed.

The labor required for a facility would be weekly inspection and routine maintenance visits (i.e., lubrication, etc.) and it is believed that a two man crew could accomplish this in 2 hours. The labor cost would be the cost of 104 hours, or at $2.50/hour, $260/year.

Maintenance supplies, replacement parts, and maintenance labor (in addition to operation-routine maintenance labor) should not exceed 1% of the facility cost per year. We have no long-term experience on the screen life at high differentials, however, it is believed that the original screen will serve for 10 or more years in stormwater service. The cost of rescreening a 10 x 10 is about $5,000. Our experience over a 3 year period has indicated a maintenance cost of less than 1% of facility cost, even if a screen change every 10 years is anticipated.

In summary, the annual cost of a facility having 490 sq ft of open submersible area (capable of treating 45 cfs) would be:

Capital charge @ 10% of installed facility cost (less land and engineering)	$19,500
Utilities - electric power and city water	200
Routine labor	250
Maintenance and supplies @ 1% of installed facility cost	1,950
	$21,900

The annual cost of installing and operating a dual 10 x 10 microstrainer facility is $22,000/year. Such a facility will accept 92 cfs and treat 46 cfs. Depending on conditions previously discussed, such a facility would serve a drainage area of from 24 to 240 acres.

The suspended solids removal performance of a microstrainer on stormwater follows a pattern that will seem strange to engineers accustomed to other liquid-solid separation techniques such as settling or granular bed filtration.

A large portion of the first increment of solids applied to the screen leak through before the mat is established. Most of subsequently applied solids are retained as shown in Figure 5. Thus, those conditions that contribute to high solids loading; i.e., high potential pollution make for high removals. These conditions are high flow rate, high stormwater solids concentration and low drum speed. It may be repeated that the higher the flow rate and the higher the influent solids, the lower the effluent solids. This latter relation is shown in Figure 6 and Figure 7.

The suspended solids in the stormwater at our site exhibited a surprising

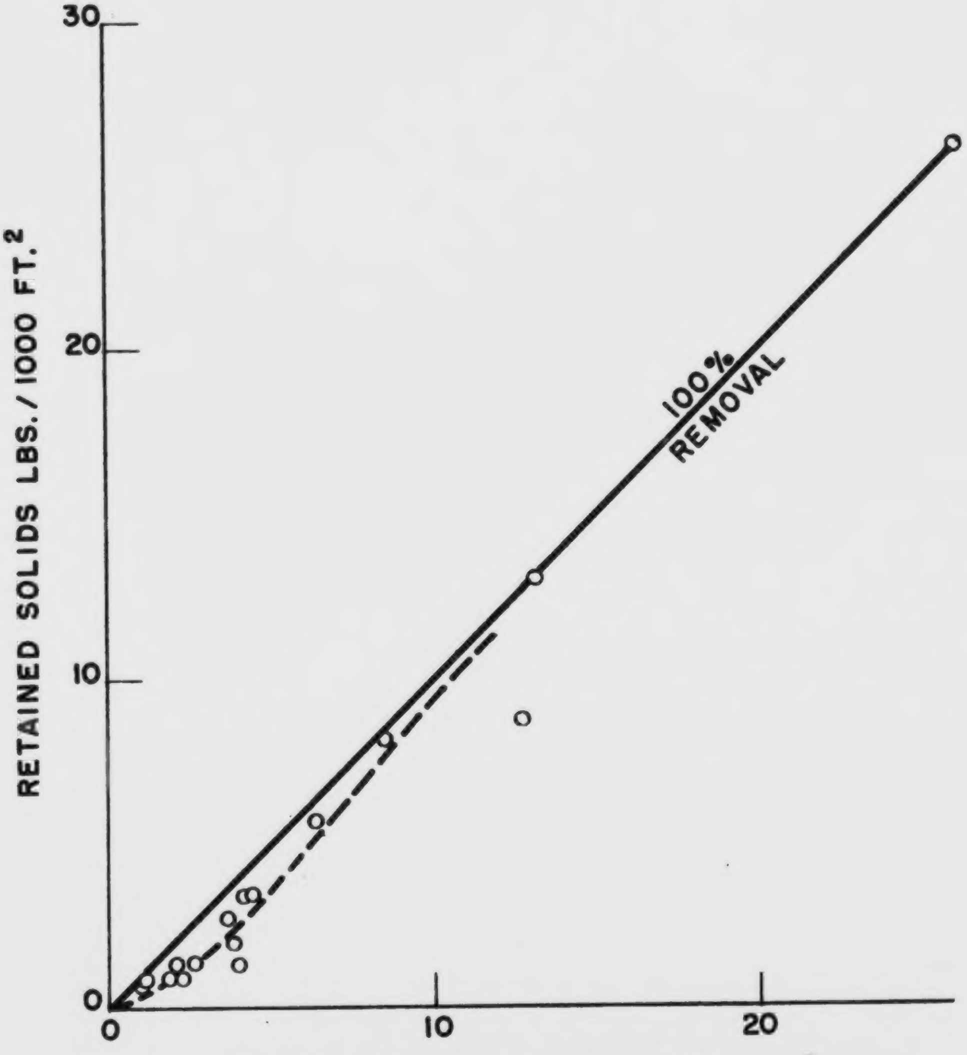


FIGURE 5
STORM WATER SOLIDS APPLIED VS.
STORM WATER SOLIDS RETAINED

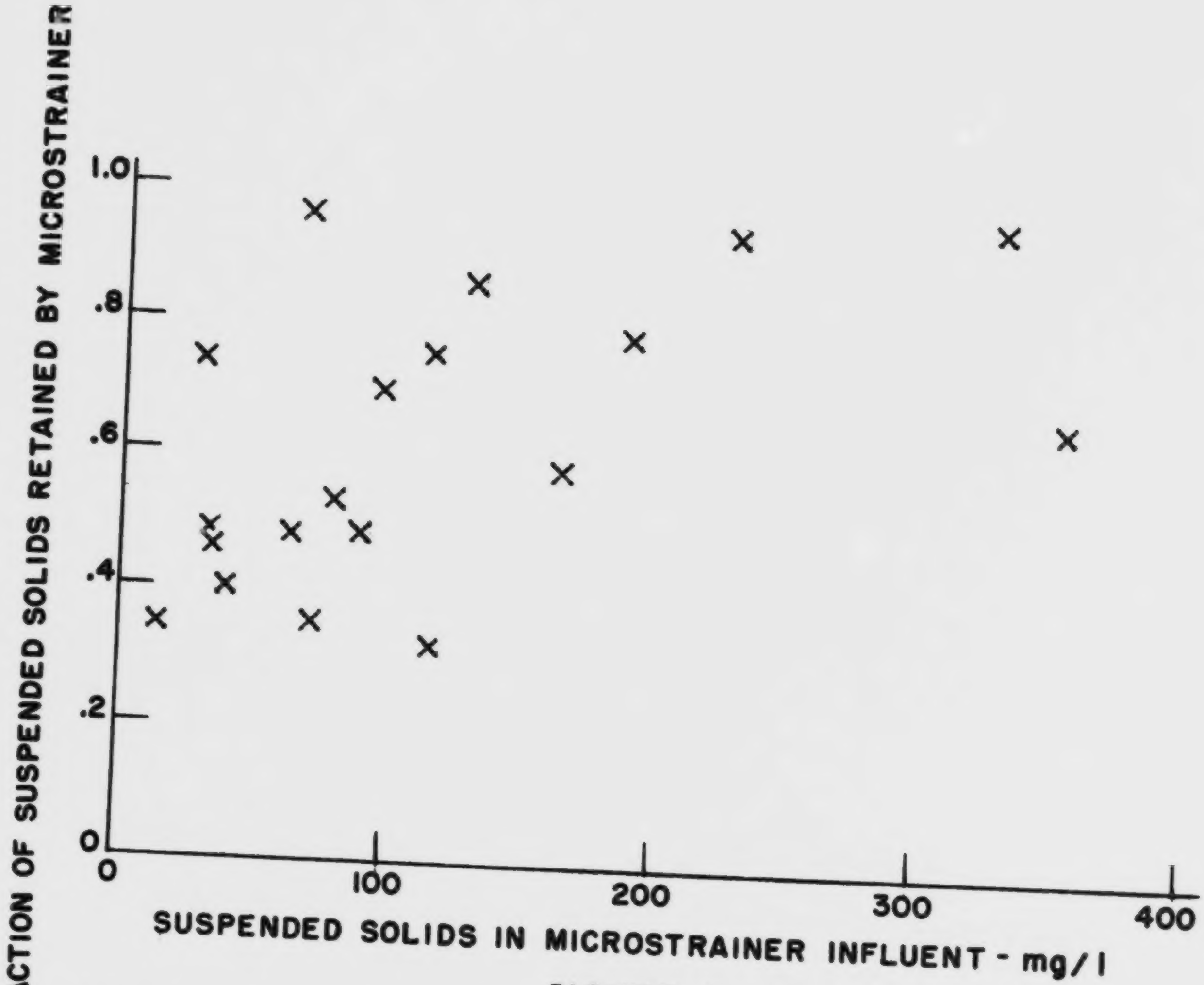


FIGURE 6
INFLUENT SUSPENDED SOLIDS FRACTION SOLIDS RETAINED

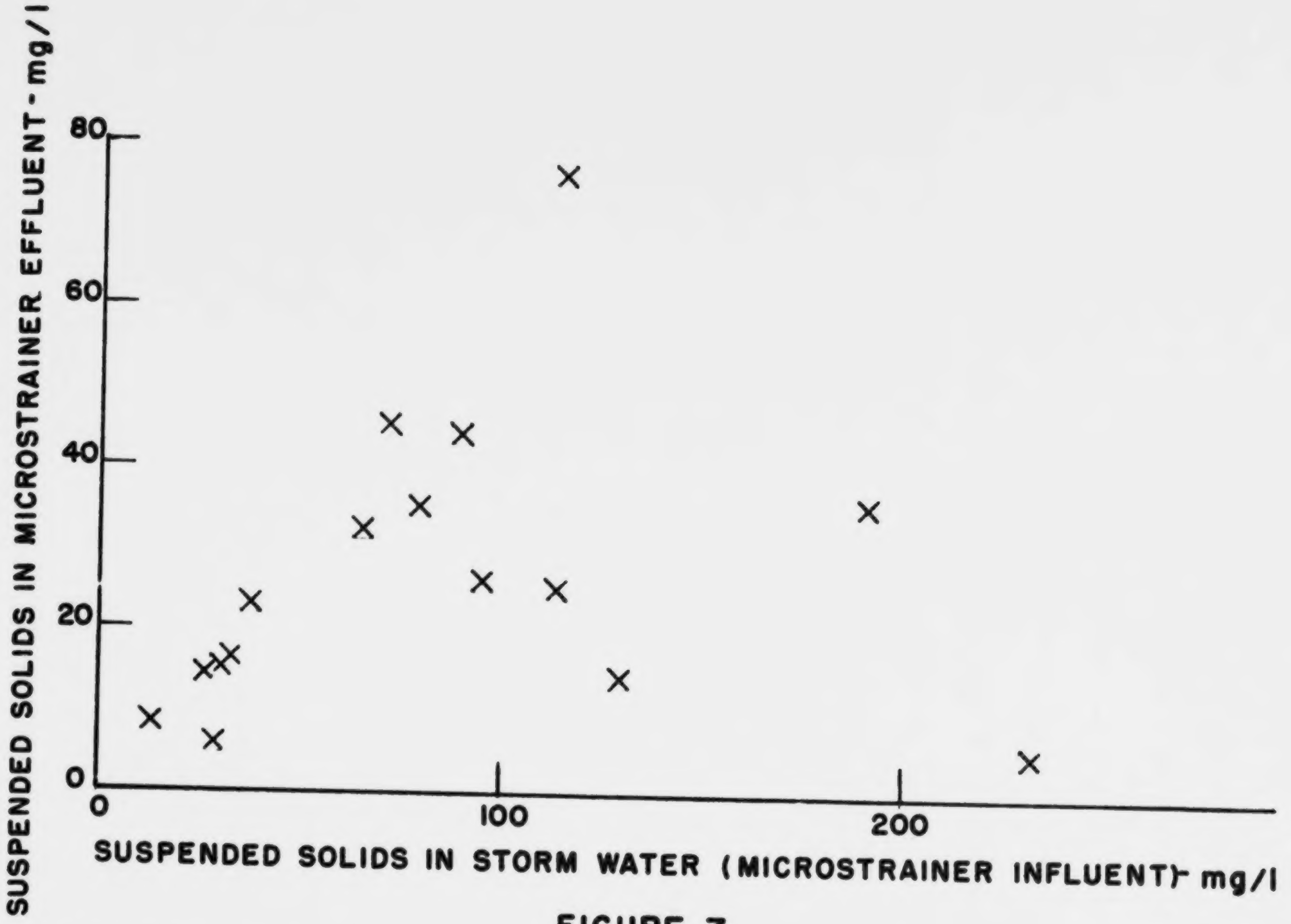


FIGURE 7
INFLUENT SUSPENDED SOLIDS VS. EFFLUENT SUSPENDED SOLIDS

characteristic. The greater the concentration of solids the easier they were strained out. The permeability parameter is the flow rate possible at unit head loss; i.e., one inch of water head loss per inch of mat thickness. The units of this parameter, borrowed from oil well practice, are inconvenient for Microstraining since buildup consists of mats of a few thousandths of an inch. In any event, this permeability is a measure of the flow capacity of the machine within the differential limitation imposed by the screen strength.

In summary, we have found in two studies totaling about 22 months of operation at one site that the microstrainer will reduce suspended solids from 50-700 mg/l down to 40-50 mg/l at flow rates of 35 to 45 gpm/ft^2 of gross submerged screen area; i.e., 42-54 gpm of unimpeded submerged area. These flow rates have been routinely achieved within an arbitrary limiation of 24" of water differential between inlet and outlet liquid levels.

The removal of organic and other oxygen demanding material is shown on Table 2 to be 25-40%. This removal is confirmed by BOD_5, COD and TOC measurements performed by the Standards Methods with and without a maceration pretreatment in a Waring Blender. The advantage of this pretreatment is covered in the formal report on this work.

The Microstraining had little or no effect on the coliform content of the stormwater.

Table 2

Effect of Microstraining on Organic Matter as Indicated by Several Test Methods

Change in Content of Organic Matter by Microstraining	BOD_5		COD		TOC	SVS	DVS
	raw	macerated	raw	macerated	macerated	raw	raw
No Changes[x]							
Number of Instances	3	3	2	1	3	5	6
Increase							
Number of Instances	1	1	2	+2	1	0	1
Average of Increases mg/l	13	14	2500	4010	2300	0	33
Average of Increases %	100	100	350	260.	250	0	100
Decrease							
Number of Instances	3	5	5	7	4	4	1
Average Decrease mg/l	129	310	516	380	126	37	33
Average of Decreases %	35	37	24	32	38	64	67

[x] All changes in concentration upon passage through microstrainer of 10 mg/l or 10% or less are considered no change.

The advantages of the Microstraining technique for suspended solids removal are:

1. Instant readiness and low residence volume permit simple automation for unattended facilities at remote locations.
2. Instant readiness and very high flow rate capability/unit equipment cost permits installation without flow equalization basins.
3. The low head loss - 3 ft - through the entire Microstraining facility will generally eliminate the need for repumping.
4. The removal performance of Microstraining, where highest removals, both absolute and percentage-wise, are achieved at highest flow rates and highest suspended solids loadings, is particularly suitable for the conditions existing in combined sewer overflow service.
5. The excess flow bypass is an integral part of a microstrainer facility and eliminates the need for this necessary feature as an appendage.
6. The very high flow rate capability and low residence volume permit Microstraining to be the lowest cost solids removal technique - less than $500/year per cfs capacity.

ACKNOWLEDGEMENT

This work was conducted with the City of Philadelphia in two phases, (1) under a contract from the Environmental Protection Agency to the Cochrane Division of the Crane Co., and (2) under an EPA grant to the City of Philadelphia. The efforts of City personnel were under the general direction of Carmen Guarino, Water Commissioner, with William Wankoff and M. Lazanoff, serving as Project Director and Laboratory Director. J. Radzuil headed the City's R and D Department who also lent valuable assistance.

The assistance and guidance of these people are gratefully acknowledged.

The overall guidance and helpful advice of Richard Field, Project Officer, EPA, Edison, New Jersey, were most valuable.

SECTION VI

HIGH-RATE MULTI-MEDIA FILTRATION

by

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GENERAL

The nature of combined sewer overflow, i.e., a highly pollutional, high volume discharge, requires a relatively high rate treatment process for economical pollution control. Deep bed, high rate filtration, a new development in the field of industrial wastewater treatment, has demonstrated favorable cost-efficiency factors when dealing with high volume wastewater discharges, especially where suspended solids comprise one of the principal contaminants. Thus, it was felt that such a process, which currently has significant applicability and usage in the steel industry, might prove an effective and efficient solution to the treatment of combined sewer overflows.

To evaluate the applicability and effectiveness of the high rate filtration process in removing contaminants from combined sewer overflows, a testing program was undertaken at Cleveland's Southerly Wastewater Treatment Plant, beginning in 1970. The work was undertaken by Hydrotechnic Corporation, Consulting Engineers, New York, New York, under the sponsorship of the Office of Research and Monitoring, USEPA.

The City of Cleveland ranks seventh in the nation in total area served by combined sewers (44,000 acres), and is fourth in population served by combined sewer systems (1,000,000 persons). As can be expected, Cleveland has a very serious problem of combined sewer overflows.

TESTING PROGRAM

The two major process units or equipment units in the proposed treatment system are the drum screen and the deep bed, high rate filter. The function of the screen is to remove coarser material (fibrous type, etc.) that would impede the filtration operation. Construction of a full scale treatment plant employing the process sequence under study would require design parameters for the screen and for the filtration process. The major criteria for the screen are screen type, screen mesh, and hydraulic loading.

The filtration system, which is the heart of the overall process sequence, can be characterized and described by the following parameters:

- Media composition
- Media depth
- Filtration rate
- Coagulant addition
- Length of filter run
- Head loss
- Backwash water volume
- Backwash procedure

A definition of these elements allows the construction of a full scale facility.

Testing equipment at Southerly included a drum screen, two 5,000 gallon storage tanks, lucite filter columns of four (4) and six (6) inch internal diameter, and chemical and polyelectrolyte feed equipment. (Figures 1 and 2)

The testing program evaluating the filtration components of the proposed system was conducted primarily in two phases: first, evaluation and selection of system media and filtration rates, and secondly, optimization of the filtration process via coagulants and polyelectrolyte addition prior to filtration.

Figure 1
Lucite Filter Columns

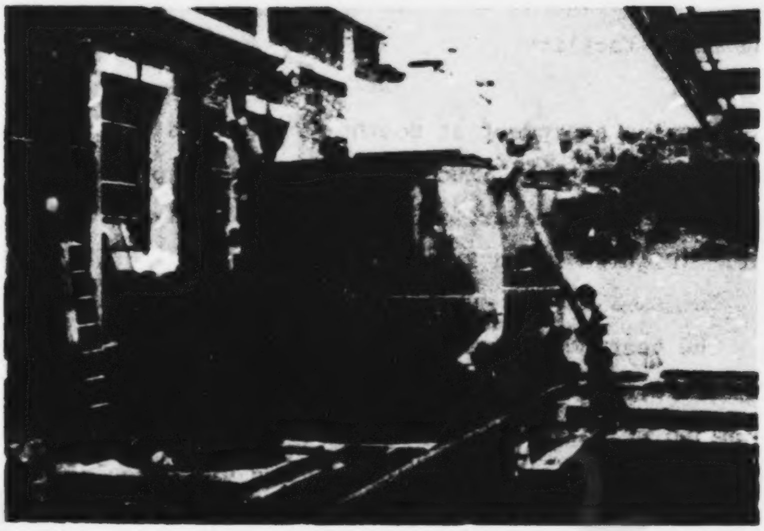

Figure 2
Drum Screen and Storage Tanks

filter run, and backwash procedure.

TEST RESULTS

The recommended system is a drum screen (No. 40 mesh screen element) followed by a deep bed, dual media filter (five feet of No. 3 anthracite over three feet of No. 612 sand). Sixty-nine pilot filtration runs were performed in 1970 and 1971 utilizing this system. Polyelectrolyte feed is an essential and critical part of the system to achieve optimum treatment efficiencies. Data utilizing coagulants ahead of filtration showed inconsistency in treatment efficiencies and at the present stage of development, polyelectrolyte feed alone appears optimum.

The proposed system, with addition of appropriate polyelectrolyte, achieved the following treatment performance:

Filtration Rate (gpm.sq ft)	Average Removals(%)		
	Suspended Solids	BOD	Phosphorous
8	96	43	66
16	95	40	57
24	93	40	46

The average influent suspended solids concentration ranged from 50 to 500 mg/l and the average influent BOD concentration ranged from 30 to 300 mg/l. Effluent levels at 24 gpm/sq ft with polyelectrolyte addition were 15 mg/l suspended solids and 22 mg/l BOD, respectively. (Figures 3, 4, and 5)

HIGH RATE FILTRATION INSTALLATION

Combined sewer overflows would be conveyed from an automated overflow chamber, or chambers (in case the centralized filtration system is for many overflow points), to a low lift pump station.

Filtration media evaluated included: four or five feet of anthracite over three feet of sand. The characteristics of the media are indicated as follows:

Media	Effective Size	Uniformity Coefficient
No. 4 Anthracite	7.15 mm.	1.42
No. 3 Anthracite	4.0 mm.	1.5
No. 2 Anthracite	1.78 mm.	1.63
No. 612 Sand	2.0	1.32
No. 48 Sand	3.15 mm.	1.27

Screen meshes tested included:

Mesh Screen Designation	Screen Opening microns/inches		Tyler Screen Scale Equivalent (mesh)	Open Area (%)
No. 3	6350	0.025	3	57.6
No. 20	841	0.0331	20	43.6
No. 40	420	0.0165	35	43.6

The filter tests were directed to determine the degree of treatment that could be achieved by using different depths and composition of filter media when operating at different flux rates, with and without the application of coagulants and polyelectrolytes. Using the results of the tests, criteria could be established to determine design parameters of full scale installations.

The principal water quality parameters carefully observed and recorded were: suspended solids, BOD, and COD. Measurements were also made on pH, temperature, total solids, settleable solids, coliforms, and total organic carbon. The laboratory analyses were performed by a local laboratory in Cleveland.

Filtration operational factors measured and recorded were: media depth and composition, flux rate, head loss, length of

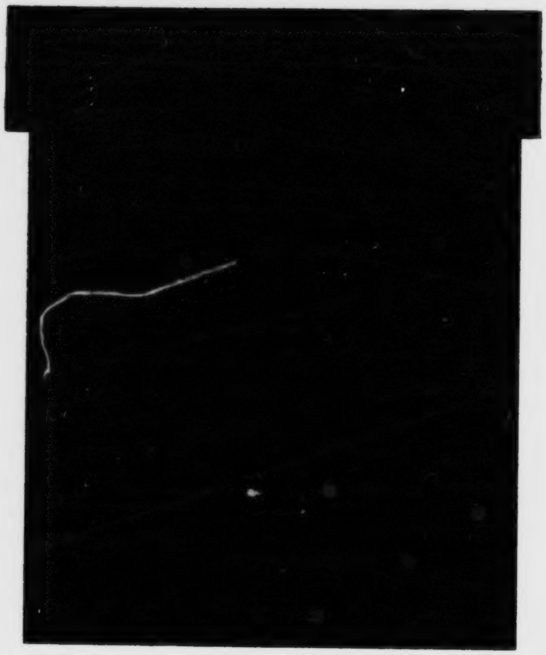

Figure 3
Filtration System
Performance--
Suspended Solids
Removal

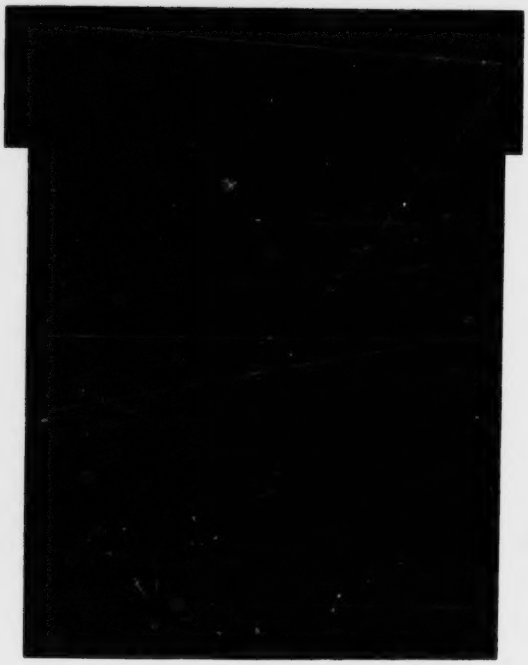

Figure 4
Filtration System
Performance -
B.O.D. Removal

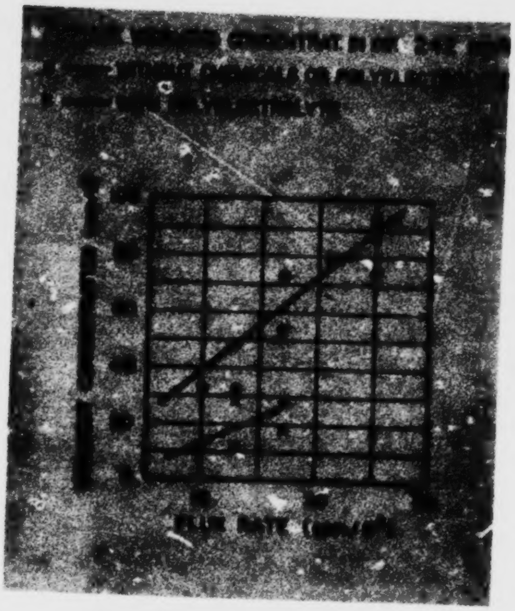

Figure 5
Filtration System Performance
Effluent Suspended Solids Quality

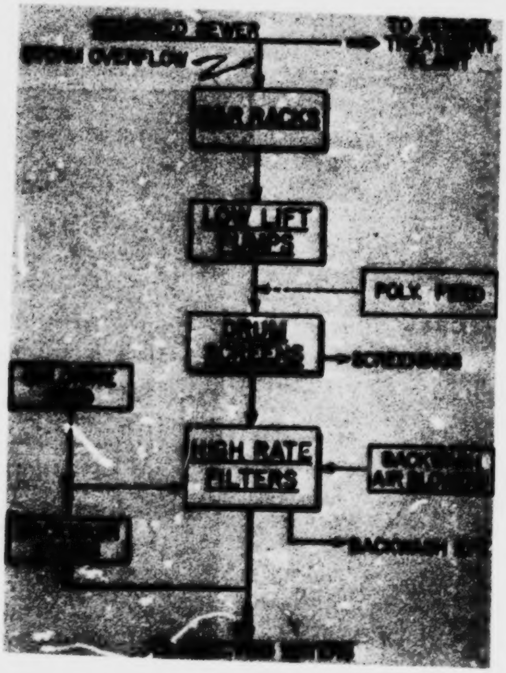
COMBINED SEWER
TO SEWAGE TREATMENT PLANT
STORM OVERFLOW
BAR RACKS
LOW LIFT PUMPS
POLY. FEED
DRUM SCREENS
SCREENINGS
HIGH RATE FILTERS
BACKWASH AIR BLOWER
BACKWASH

Figure 6
High-Rate Filtration Plant
Flow Diagram

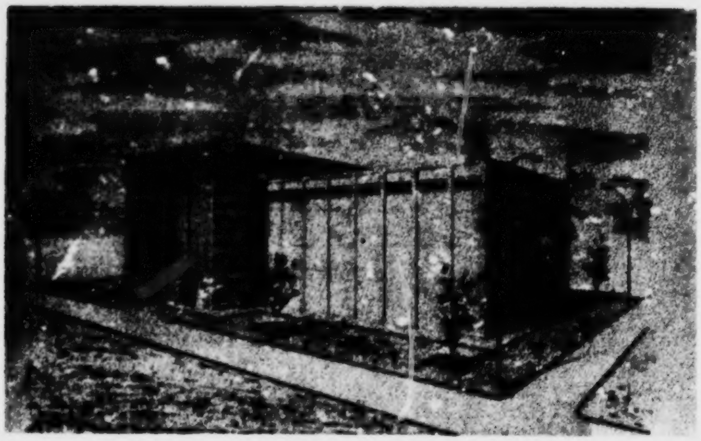

Figure 7
High Rate Filtration Installation

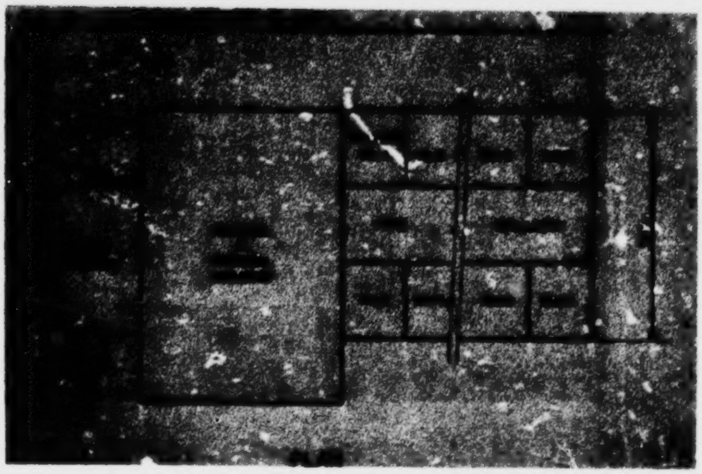

Figure 8
Plan - High Rate Filtration Installation
(100 MGD)

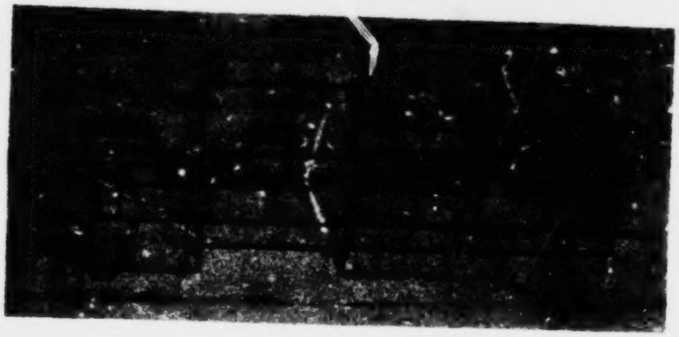

Figure 9
Longitudinal Section High Rate
Filtration Installation (100 MGD)

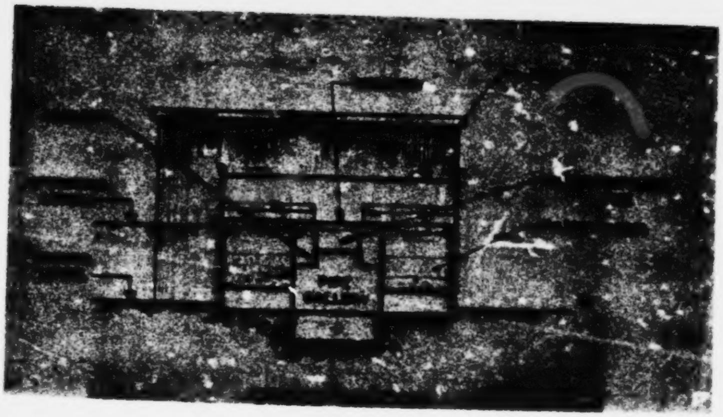

Figure 10
Cross Section - High Rate
Filtration Installation (100 MGD)

Before entering the pumping station, the combined sewer overflow would pass through a bar rack (screen) for removal of coarse materials which might cause problems in the operation, maintenance or wear of the low lift pumps. In certain locations, where consistent with local topography and sewer invert, a low lift pumping facility may not be required.

The combined sewer overflow from the low lift pump station would enter a treatment building and be delivered to drum type screening units. The wastewater would be introduced into the center of the drum type screen and would pass through the screening mesh into the influent channel to the filters. A gravity type design, i.e., open filtration units, is proposed. The water would be introduced at the top of the filter and flow downward through the filter bed. The plant effluent could be discharged by gravity to the respective receiving water body.

Filtered wastewater would serve as a source of water for backwashing filters after the overflow has attenuated to a sufficient degree. The filtration building would be provided with low pressure air blowers as a source of backwash air. Backwash pumps would be located in the filtration facilities to deliver water to the filters for backwashing. The treatment building would also include a control area, office space, a polyelectrolyte feeding set-up, and a system for adding hypochlorite to filter backwash water for the prevention of slime growth on the filter media. The operation of the high rate filtration facility would be completely automated, and could be left unattended, except for routine maintenance and periodic delivery of chemicals. In full size treatment systems, chlorine feed for disinfection could be incorporated into the filtration facilities.

Dirty backwash effluent from the filtration facilities and screenings would be directed into the interceptor running to the sanitary sewage treatment facility. The concentrated solids from the drum screening units would be passed first through a grinder, and then through a trash basket or classification device to insure that very coarse, settleable material is not returned to the sewer system. Sludge handling facilities should not be located at the filtration site, as this would prove very costly. Centralization of material handling facilities has always proved most economical; as an example, the Southerly Wastewater Treatment receives sludge from another plant in Cleveland.

For filter backwashing, two types of process control should be considered: the first parameter would be total head loss through filter bed, and the second would be effluent suspended solids concentration.

For measuring the filter head loss, each filter would be equipped with a differential pressure transmitter to continuously sense the loss of head across the filter and transmit a pneumatic signal linearly proportionate to this head loss to a central control panel. When the filter head loss would reach a preset value, the differential pressure switch associated with the filter would be actuated. A contact in this switch would open a stepping switch circuit and the filter would start to backwash.

An alternate, filter backwash control could be achieved with an effluent suspended solids monitor. A continuous reading, light scatter type suspended solids meter would be installed in each filter effluent pipe to continuously measure the suspended solids concentration and transmit the reading to a recorder at a central control panel. When the filter breakthrough would suddenly take place and the suspended solids concentration indicator would reach a preset level, then a micro switch would be activated and an alarm would be initiated. The operator would check the filter performance condition and start to backwash the filter.

Principal advantages of the proposed system are: high treatment efficiencies, automated operation, and limited space requirements as compared with alternate flotation or sedimentation systems.

COST DATA

Estimated total construction costs (ENR=1470) of a filtration plant for treating combined sewer overflows range from $830,000 for the 25 MGD capacity to $3,754,000 for 200 MGD capacity at design rate of 24 gpm/sq ft.

Estimated annual cost data ranges from $97,270 per year for a 25 MGD capacity plant to $388,210 per year for a 200 MGD capacity plant. Annual treatment costs utilizing the high rate filtration process are due primarily to interest and amoritzation charges, and are less affected vby the volume of combined sewer overflow to be treated annually.

These costs do not include disposal of waste screenings and filter backwash since the proposed system would discharge these to the municipal sewage treatment plant. Assuming an average of 200 mg/l of solids removed and a combined sewer overflow treatment plant operation of 300 hours per year, solids processing and disposal costs incurred by the municipal sewage treatment plant could range from 3 to 35 percent of the total annual charges for the combined sewer overflow treatment facility.

DUAL PURPOSE OF UTILIZATION OF HIGH RATE FILTRATION PROCESS

The selected media for combined sewer overflow treatment was also evaluated in terms of its capacity for polishing secondary effluent under another research contract. Test data has confirmed the applicability of this combined sewer overflow media to reducing suspended solids, BOD, and phosphate to low residuals.

In Cleveland, the total duration of the overflows from the combined sewer system is approximately 300 hours per annum. This indicates the possibility of utilizing dual purpose treatment plants based on the high rate filtration process. Such installations would treat combined sewer overflows when they occur, and in between such periods, for over 95 percent of the time, the filtration process would treat other wastewaters depending on the location of the process.

For a high rate filtration process for combined sewer overflow treatment located in the area of the domestic wastewater treatment plant, the filtration process can be utilized for polishing the treatment plant effluent as well as to protect the effluent quality during plant overloading or process malfunction.

The economical benefits of such dual purpose utilization of the high rate filtration process should not be overlooked.

SECTION VII

SCREENING/DISSOLVED - AIR FLOTATION TREATMENT OF COMBINED SEWER OVERFLOWS

by

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Introduction

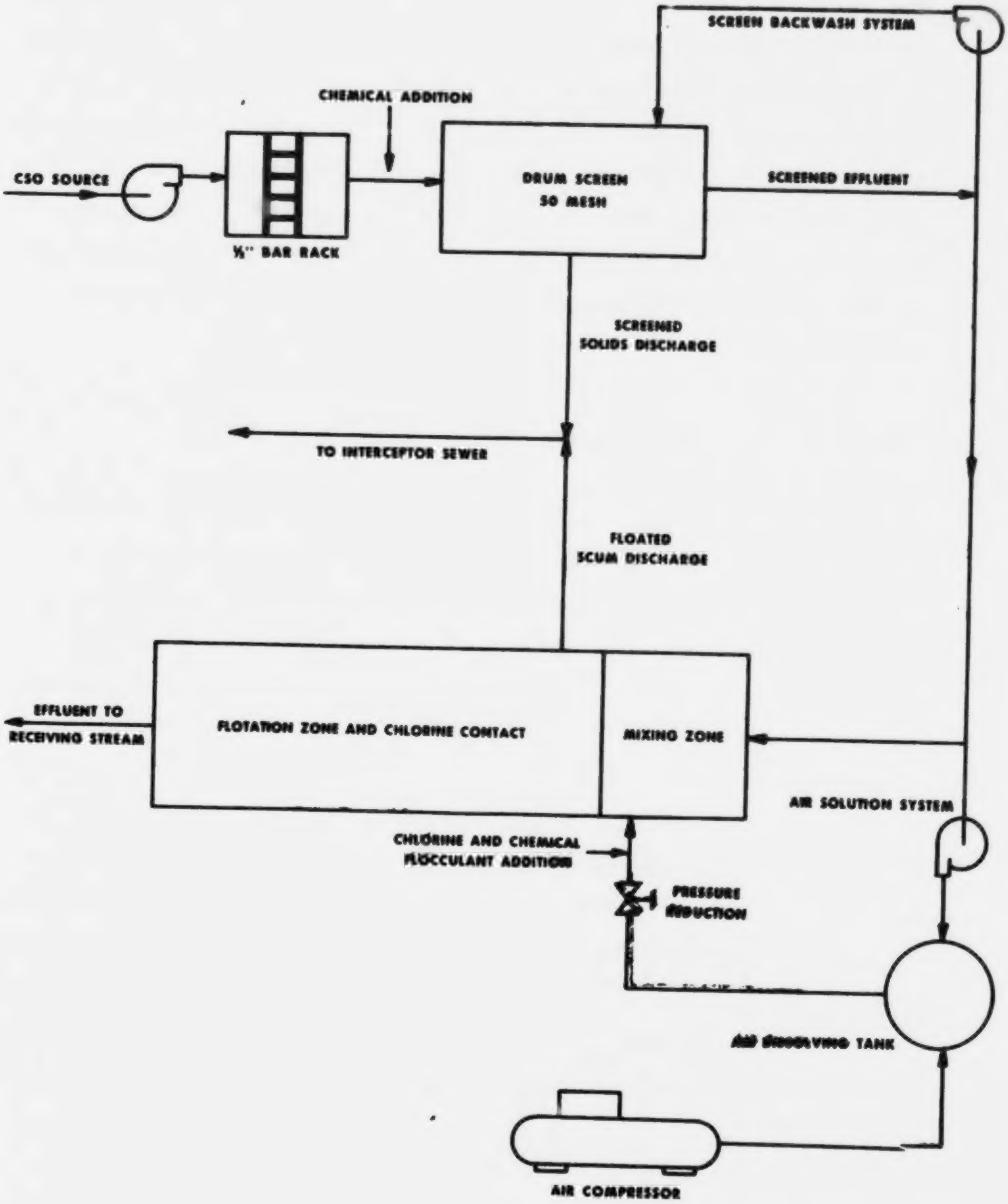
The problem of combined sewer overflows (CSO) has been recognized as a significant pollution problem in recent years (1). Large amounts of untreated pollutants find their way into our water courses through this route. The abatement methods dealing with this problem are sewer separation, storage, treatment, or a combination of these. The cost of separating the sewers is prohibitive and this method is not considered as an economical solution to the problem. A great deal of literature has been published since 1964 which describes the characteristics of (CSO) (2). Based on the data published, it has now been established that a major portion of the pollutional substances in CSO is particulate in nature. This indicates that an efficient solid/liquid separation process can be expected to provide an effective treatment of CSO. It was the mission of the Environmental Sciences Division of Rexnord Inc. to develop an effective and economical solid/liquid separation process under a program sponsored by the U.S.Environmental Protection Agency.

A combined sewer outfall near Hawley Road in the west-central portion of Milwaukee, Wisconsin was selected as a source of combined sewer overflow for the bench scale studies. This outfall services a 495 acre residential area. It was determined that approximately 42% of the area was impervious, i.e. streets and parking areas, house roofs etc. The calculated value of the runoff coefficient was 0.40 and it compares well with the values reported in the literature (3). The drainage area comprises of mostly one and two family dwellings with an estimated density of 35 people per acre. No manufacturing industries are located within the drainage area except some small business shops. Bench scale tests were conducted on 14 separate overflow samples to define the quality of the Hawley Road outfall and to evaluate the various potential treatment processes. The evaluatory tests included screening with various sized media, chemical oxidation, flotation and disinfection. It was determined from these tests that chemical oxidation of the raw CSO did not appear technically and economically feasible (4). However, the results of the screening and dissolved-air flotation tests were encouraging. These tests served as the design basis of a 5 MGD test facility at the Hawley Road outfall utilizing screening and dissolved air flotation.

Design of the Treatment System

The process schematic of the proposed treatment system is shown in Figure 1. The raw overflow is pumped from the sewer to a half inch manually cleaned bar rack. The purpose of the bar rack is to remove large objects which may clog or damage the finer screen downstream. The flow then enters a 50 mesh (approximately 300 micron) drum screen. The basic screen is fabricated from mild carbon steel while the screening media is a 304 stainless steel. The screen is an octagonal shaped drum with an effective diameter of 7.5 ft. and 6 ft. length. The total screen area is 144 sq. ft. with wetted screen area ranging between 72 and 90 sq. ft. depending upon the head loss across the screen. The design hydraulic loading for the screen is 50 gpm/sq. ft. and a maximum head loss capacity of 14 inches. The drum speed can be varied in the range of 0.5 to 5.0 rpm.

Screened water is used to backwash the screen. The solids which are removed from the screen are collected in a hopper and are then routed to the sanitary sewer. The screened effluent is split into two portions. A major portion of the flow goes directly to the flotation tank while the remainder of the flow


SCREEN BACKWASH SYSTEM
CHEMICAL ADDITION
CSO SOURCE
½" BAR RACK
DRUM SCREEN
50 MESH
SCREENED EFFLUENT
SCREENED
SOLIDS DISCHARGE
TO INTERCEPTOR SEWER
FLOATED
SCUM DISCHARGE
EFFLUENT TO
RECEIVING STREAM
FLOTATION ZONE AND CHLORINE CONTACT
MIXING ZONE
AIR SOLUTION SYSTEM
CHLORINE AND CHEMICAL
FLOCCULANT ADDITION
PRESSURE
REDUCTION
AIR DISSOLVING TANK
AIR COMPRESSOR

SCREENING/FLOTATION FLOW DIAGRAM

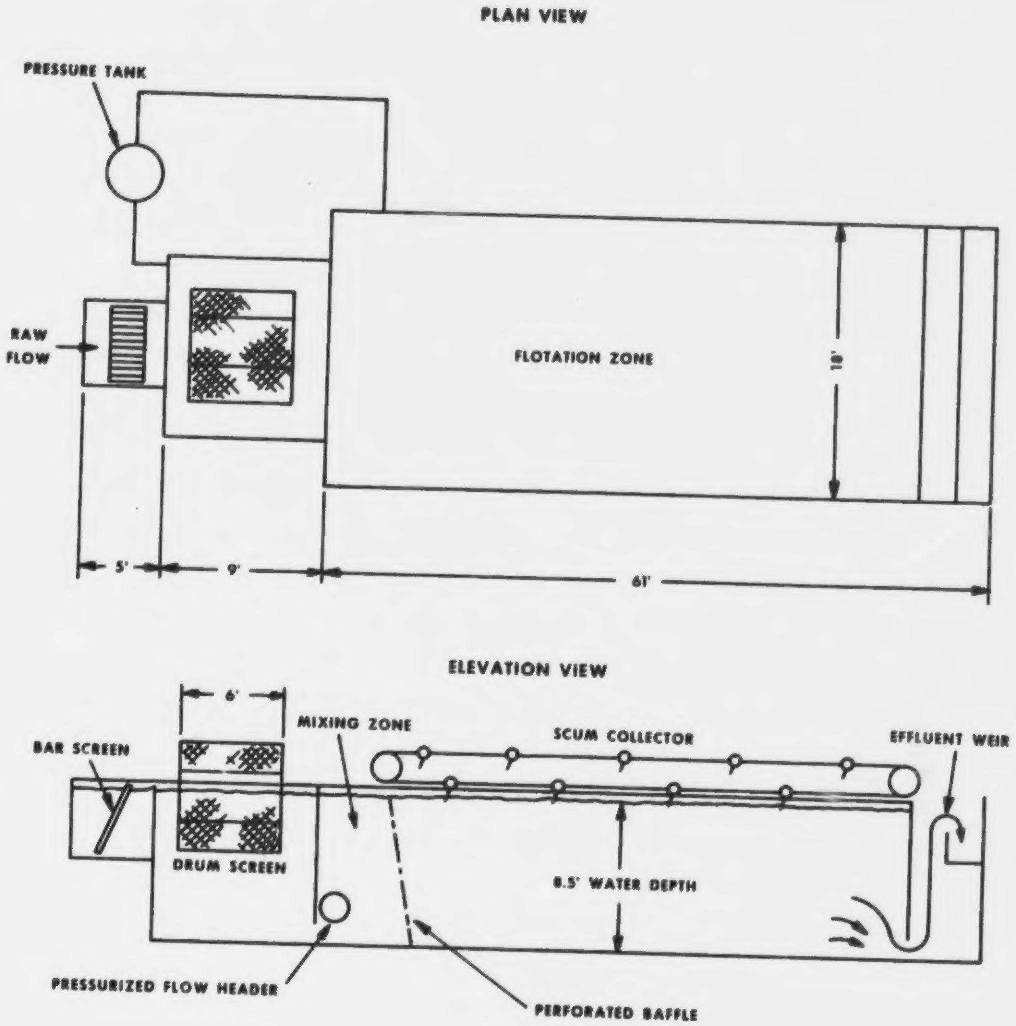
FIGURE 1

(approx. 20%) goes to a pressure tank where it is mixed with air under pressure (approx. 50 psi). The pressurized air-water stream is then brought into contact with the bulk of the raw flow at atmospheric pressure in a mixing zone. The dissolved air comes out of solution in the form of tiny bubbles (50-100 micron size) in the mixing zone and these bubbles attach themselves to the suspended matter in the waste water. The mixed flow then passes through a distribution baffle and into the flotation tank where solid/liquid separation occurs. The scum which floats to the top is then scraped into a trough via skimmers and is routed to the sanitary sewer. The treated effluent is discharged to the Menomonee River.

The main details of the treatment system are shown in Figure 2. Flexibility was provided in the design so that the flotation zone could be segmented for evaluating various hydraulic overflow rates. Chemical flocculants when utilized were added to the raw waste as it entered the drum screen or in the pressurized flow stream after the pressure reduction valve. Chlorine was also added in the pressurized flow stream for disinfection of the CSO. The entire system was automated and was put into operation by sensing a pre-set level of the waste water in the sewer.

Operation of the Demonstration System

The system was operated on 55 separate combined sewer overflows during 1969 and 1970. The quality characteristics of these overflows are seen in Table 1. About 20 percent of the overflows exhibited the first flush phenomenon, which was either caused by high rainfall intensity or a length of time greater than four days between overflows. After the first flush diminished, the quality of the overflow was remarkably constant for each storm. The 95% confidence ranges for the extended overflows were only about 10-


PLAN VIEW
PRESSURE TANK
RAW FLOW
FLOTATION ZONE
10'
5'
9'
61'
ELEVATION VIEW
6'
BAR SCREEN
MIXING ZONE
SCUM COLLECTOR
EFFLUENT WEIR
DRUM SCREEN
8.5' WATER DEPTH
PRESSURIZED FLOW HEADER
PERFORATED BAFFLE

DEMONSTRATION SYSTEM DETAILS

FIGURE 2

TABLE 1

COMBINED SEWER OVERFLOW CHARACTERISTICS AT HAWLEY ROAD[1]

Analysis	First Flushes[2]		Extended Overflow[3]
Total Solids (mg/l)	861 ± 117		378 ± 46
Total Volatile Solids (mg/l)	489 ± 83		185 ± 23
SS (mg/l)	522 ± 150		166 ± 26
VSS (mg/l)	308 ± 8.3		90 ± 14
COD (mg/l)	581 ± 92		161 ± 19
BOD (mg/l)	186 ± 40		44 ± 10
Total Kjeldahl Nitrogen (mg/l)	17.6 ± 3.1		5.5 ± 0.8
pH	7.0 ± 0.1		7.2 ± 0.1
Total Coliform (individuals/ml)	142 x 10^3 ± 108		62.5 x 10^3 ± 27
Dissolved COD/Total COD[4]	--	0.34 ± 0.04	--

[1] Ranges shown at 95 percent confidence level.

[2] Represents 12 overflows.

[3] Represents 44 overflows.

[4] Represents 34 overflows.

15% of the mean value as compared with 20-25% for the first flush data. The dissolved organic fraction (measured as chemical oxygen demand) was approximately one third of the total organic load in the raw combined sewer overflow. This showed that a large portion (2/3 of the total) of the organic pollutants was of a particulate nature which would be amenable to treatment via screening/dissolved-air flotation.

The variables evaluated during operation included hydraulic loading and drum speed for the screening operation, and surface overflow rate, pressurized flow rate, operating pressure, and flocculant dosages for the flotation system. The optimum operating conditions based on the treatment of 55 CSO are given in Table 2. The optimum solids loading rate at a drum speed of 4.7 rpm and a head loss of 12" was 1.2 pounds of dry solids removed per 100 sq. ft. of screen area. This loading could possibly be increased by increasing the allowable head loss differential. The hydraulic throughput rate was in the range of 40-45 gpm/sq.ft. This rate again can probably be increased depending upon solids loading. It was found that no statistical difference could be shown in the removal efficiencies by increasing the pressurized flow rate up to 45 percent of the raw flow, or by increasing the operating pressure to 60 psi. A pressurized flow rate of 20% of the raw flow at 50 psi was recommended for future designs. The air usage was approximately one cfm per 100 gpm of pressurized flow. The overflow rate at which removal efficiencies were satisfactory and the capital cost still reasonable was 3.3 gpm/sq.ft. Floated scum concentrations generally ranged between 0.7 and 1.4% of the raw flow. The chemical flocculants utilized during this study were $FeCl_3$ and a cationic polymer (C-31, Dow Chemical Co.). The selection of these chemicals was based on the results of a series of bench scale jar tests. The optimum chemical dosages were found to be 20 mg/l $FeCl_3$ and 4 mg/l of C-31.

TABLE 2
OPTIMUM OPERATIONAL CONDITIONS

Characteristics	Operational Condition
Screening	
Backwash	0.7 - 1.0% raw flow
Head Loss	12 in. water
Rotation Speed	4.7 rpm
Submergence	50 - 63%
Hydraulic Throughput Rate	40 - 45 gpm/sq. ft.
Flotation	
Floated Scum	0.75 - 1.41% raw flow
Pressurized Flow	20% raw flow
Operation Pressure	50 psi
Overflow Rate	3.3 gpm/sq. ft.
Chemical Dosage	20 mg/l $FeCl_3$ 4 mg/l cationic polyelectrolyte

The performance of the 50 mesh screen alone is summarized in Table 3. The pollutant removals (measured in terms of suspended solids, volatile suspended solids, COD and BOD) ranged between 33-39% for the first flushes and between 26-34% for the extended overflows. The slightly higher removal efficiencies for the first flush overflows is probably a result of the screening-filtration phenomenon that occurs during these high pollutant loading periods.

The total removal efficiencies for the combined screening/flotation system are shown in Table 4. The pollutant removals ranged between 35-48% without flocculating chemicals. However, the removal efficiencies were significantly enhanced on the addition of flocculating chemicals and ranged between 57-71%. Removals during the first flushes were similar to the results for extended overflows with chemical addition. The average effluent quality experienced with chemical addition and that can be expected via screening/flotation treatment is shown in Table 5. These values compare favorably with many secondary sewage treatment effluents.

Future Design Considerations

The data presented so far had been based on the results of two operational seasons, 1969 and 1970. Research was continued on this treatment facility during 1971 to obtain additional design data for the optimization of the screening and dissolved-air flotation processes in order to improve upon the effluent water quality of the treated combined sewer overflows.

Laboratory bench scale tests have indicated that changing the split flow mode of dissolved-air flotation to effluent recycle mode of operation may enhance the effluent water quality significantly. This change may require the operation of the flotation

TABLE 3
PERCENT POLLUTANT REMOVALS BY SCREENING*

Characteristics	First Flushes	Extended Overflow
SS	36 ± 16	27 ± 5
VSS	37 ± 18	34 ± 5
COD	39 ± 15	26 ± 5
BOD	33 ± 17	27 ± 5

* Values given at the 95 percent confidence level.

TABLE 4
PERCENT POLLUTANTS REMOVALS BY SCREENING/FLOTATION TREATMENT*

Characteristic	First Flushes	Extended Overflows	
		Without Chemicals	With Chemicals
SS	72 ± 6	43 ± 7	71 ± 9
VSS	75 ± 6	48 ± 11	71 ± 9
COD	64 ± 6	41 ± 8	57 ± 11
BOD	55 ± 8	35 ± 8	60 ± 11
Nitrogen (total Kjeldahl)	46 ± 7	29 ± 8	24 ± 9

* Values shown in a 95 percent confidence range.

TABLE 5
EXPECTED AVERAGE EFFLUENT QUALITY AT HAWLEY ROAD

Analysis	Value (mg/1)
SS	48
VSS	26
COD	69
BOD	20
Nitrogen (total Kjeldahl)	4.2

system at reduced overflow rates and could therefore increase the flotation area requirements by approximately 20%.

Also, several other chemical flocculant combinations have shown promise over the ferric chloride - C-31 polymer combination utilized during the 1969 and 1970 operational seasons. Use of powder activated carbon along with screening/dissolved-air flotation has also shown some merit. The economics of these concepts for an optimum cost benefit relationship still need evaluation. These evaluations are a part of the proposed modifications to the Hawley Road treatment facility. It is anticipated that these considerations will be evaluated on the modified Hawley Road treatment facility during the 1973 operational season.

Racine Root River Project

Encouraged by the promising results of the Hawley Road demonstration facility, a search was made to find a site where the feasibility of utilizing screening/dissolved-air flotation could be demonstrated on a full scale for the treatment of combined sewer overflows. The City of Racine, Wisconsin was indicated to be an ideal site for such a project. Racine is a city of approximately 100,000 people located on Lake Michigan, approximately 30 miles south of Milwaukee. The Root River, a stream having a mean annual discharge of approximately 100 cfs flows through the city and serves as a receiving body for runoff from much of the northern half of the city. There are approximately 700 acres of land having combined sewer systems in this area. In the 3.7 miles of Root River through the city, there are 36 combined sewer overflow points and 17 storm water discharges to the river. It was estimated that the cost of separation of the existing combined sewer areas in Racine would be 10-13 million dollars. The estimated cost of installing the screening/dissolved-air flotation treatment plants at the various outfalls was 4 million dollars. Thus significant savings were evident in going for the screening/dissolved-air flotation route for the treatment of combined sewer overflow problem in the City of Racine.

In April of 1970 a grant application was submitted to the U.S. Environmental Protection Agency. Under the terms of this proposal the funds would be rendered by the federal government, State of Wisconsin, and the City of Racine. The technical approach proposed for meeting the project objectives includes the following elements:

1. Quantitative measurement of the effects of treating stormwater discharges and combined sewer overflows to a selected stretch of the Root River as a function of river water quality.
2. Detailed cost/performance analysis for treatment in the selected stretch of the Root River.
3. Full-scale verification of the combined sewer/water quality mathematical model developed under EPA Grant No. S800744.
4. Application of results from Items 1, 2, and 3 above to the following determinations:
    a. Process adequacy of treatment system as an alternate to combined sewer separation in 700 acre area of central Racine, Wisconsin.
    b. Cost/benefit relations of treatment system for the subject 700 acre area in central Racine, Wisconsin.
    c. Validity of the EPA Storm Water Management Model for application to problems of any given area.
    d. Process, design, operation and application criteria for treatment method as alternate to combined sewer separation in any given area.

This proposal was approved by the City of Racine and in July, 1970 the project was approved and funded by EPA. The City of Racine became the grantee and a subcontract was awarded to the Environmental Sciences Division of Rexnord Inc. to conduct the study program. Also the consulting engineering firm of Donohue and Associates of Sheboygan, Wisconsin was retained by the City of Racine for the engineering design of this project.

An engineering study was conducted by the Rexnord Environmental Sciences Division and Donohue and Associates in cooperation with the Racine City Engineer's Office for the purpose of choosing a site which would allow for maximum CSO treatment within the project dollars. The two selected locations which contained the overflow outfalls from a large percentage of the total area and where these discharges were in close proximity to one another were:

1. Site I - South of Dodge Street between Chatham and Michigan Streets
2. Site II - South of Dodge Street between Main and Wisconsin Streets

The overflows in these locations drain from a total area of 450 acres of combined sewers. Site I was designed for a treatment capacity of 14.1 mgd and Site II for 44.4 mgd.

System Design

Two full scale SDAF systems have been installed in Racine for treatment of combined sewer overflow. The design criteria for each of the various elements is shown in Table 6. The systems have been designed for completely automatic startup, operation and shutdown.

The two systems are similar in function and differ only in design capacity. A schematic diagram of the larger system is

TABLE 6
DESIGN CRITERIA - SCREENING/AIR FLOTATION TREATMENT SYSTEM
RACINE, WISCONSIN

Item	Site #1	Site #2
Contributing area (acres)	82.5	364.2
Design Storm Intensity (inch/hour)	0.5	0.5
In-Sewer Storage (gallons)	--	600,000
Design Flow for Treatment System (MGD)	14.13	44.4
Bar Screens		
Mechanically cleaned and located Just Upstream of Pump Sump	Yes	Yes
Drum Screens		
Parallel Operation, automatic bypass to flotation tanks should all screens clog		
Number of screens	2	4
Length (feet)	7	10
Diameter (feet)	8	8
Filter Media Stainless Steel - 50 mesh, .009 inch wire		
Screen Backwash flow gpm (when operating)	210	675

TABLE 6 CONTINUED

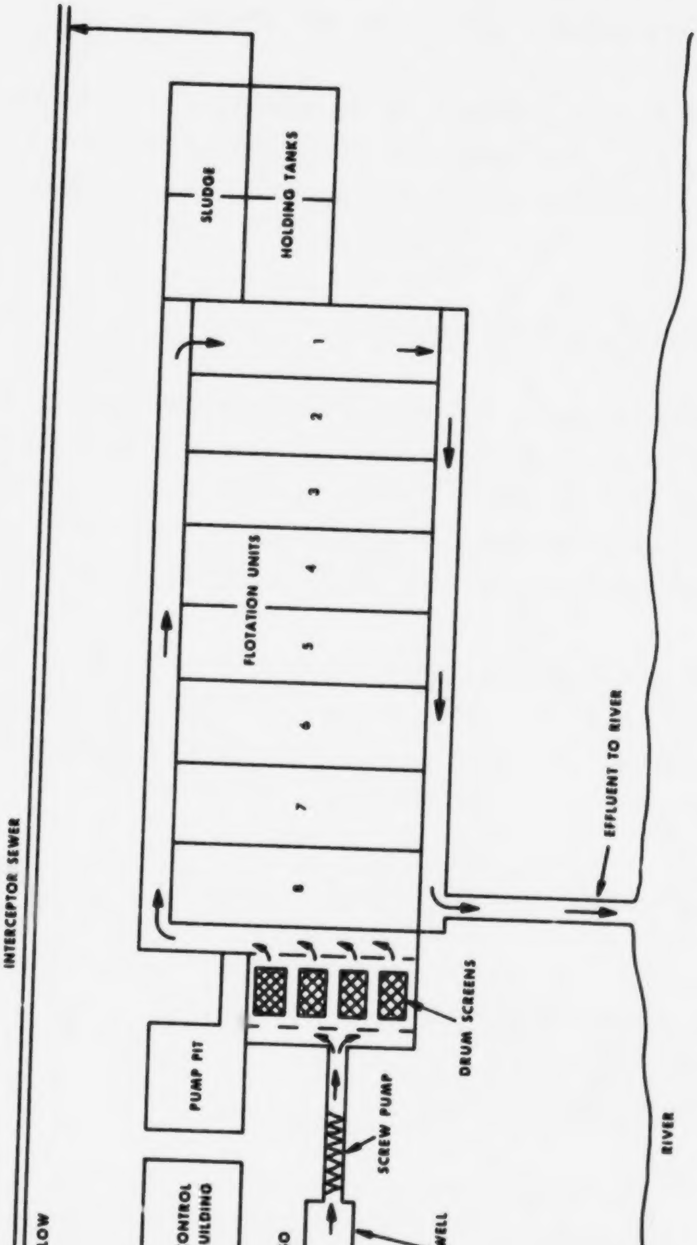
Item	Site #1	Site #2
Flotation System		
Operation - Each tank reaches 70% maximum flow before the next tank is put into use.		
Number of tanks	3	8
Surface overflow rate - gpm/ft^2	3.5	3.5
Pressurized flow - gpm/tank	650	770
Scum Removal - timer controlled Surface skimmer to scum trough - Screw conveyed to sludge holding tank		
Chemicals		
Chlorine - maximum concentration mg/l	20	20
$FeCl_3$ - maximum concentration mg/l	25	25
Polyelectrolyte - concentration Dependent on specific polyelectrolyte		
Sludge Storage		
1.5% of design flow for 3 hour duration Volume - cubic feet	3,500	11,030
Disposal to sanitary sewer by gravity Drain following storm		

shown in Figure 3. Upon sensing a high level in the overflow sewer, the system is placed into operation. Raw overflow enters the plant through a mechanically cleaned bar screen located in the wet well. A by-pass weir is provided for storm flows in excess of the design capacity. Flow entering the wet well is pumped by means of a spiral screw pump through a Parshall flume and into the screening chamber. The output of the flow recorder/totalizers are used to provide a proportional signal for pacing the chemical feed equipment. Ferric chloride is added to the wastewater upstream of the screens. Chlorine and polyelectrolyte are added downstream of the screens.

Each of the drum screens is equipped with 50 mesh stainless steel screens. The screens are backwashed at a preset headloss level. Solids removed on the screen are conveyed to a sludge holding tank by means of a screw conveyor which runs along the head end of the flotation tanks.

Effluent from the drum screens is diverted to the flotation tanks by means of a series of weirs and orifices. The inlet system is designed so that the tanks are filled in series. This enables the utilization of only as much tankage as is actually required by the storm flow. Screened effluent is used as the source of pressurized flow.

Scum produced in the air flotation tanks is skimmed to the head end of the tanks where it is conveyed to the sludge holding tanks by means of a screw conveyor. All sludge generated during a storm is held in the holding tanks until after the storm subsides and then is discharged to the interceptor sewer. At some future date it may prove fruitful to provide onsite dewatering facilities rather than return the concentrated sludge to the sewer system.


INTERCEPTOR SEWER
OVERFLOW
CONTROL BUILDING
PUMP PIT
RAW CSO
WET WELL
SCREW PUMP
DRUM SCREENS
EXCESS OVERFLOW
RIVER
EFFLUENT TO RIVER
FLOTATION UNITS
1
2
3
4
5
6
7
8
SLUDGE
HOLDING TANKS

SCHEMATIC LAYOUT OF THE TREATMENT SYSTEM FOR SITE NO. 2
FIGURE 3

The flotation tank effluent which has been chlorinated will be discharged directly to the Root River.

Following a storm all of the sludge, as well as the contents of the flotation tanks will be discharged to the adjacent sanitary interceptor sewer. The system will then be ready for the next storm.

Special Considerations

Certain special considerations have been made in order to insure optimum use of the system. A floodgate was installed in one of the overflow sewers to provide approximately 600,000 gallons of in-system storage. This storage capacity will be utilized when the treatment facility reaches full capacity.

In addition, the system has been equipped to be completely self-draining. This will enable use of the system during periods of snow melt and cold weather. A roof has also been provided to prevent floc breakup during heavy rains.

Costs

The cost for the Racine SDAF system is $30,000 per mgd installed capacity. A detailed cost breakdown is given in Table 7.

Racine Program

A two year system evaluation and optimization is scheduled to begin on April 1, 1973. The intent of this program is to fully evaluate the installed facility, validate the EPA Stormwater Management Model and determine the effect of the system on water quality in the Root River.

TABLE 7
COST OF SCREENING/DISSOLVED AIR FLOTATION

Capital Costs

Cost per MGD Capacity	$30,000
Cost per Acre*	$ 3,900

* Based on 0.5"/hour runoff rate

Operating Costs

	¢/1000 gallons
Power	0.54
Chemicals	2.51
Maintenance	0.04
TOTAL	3.09¢/1000 gallons

Based on plant capacity of more than 30 MGD and 40 hours per month operation.

Acknowledgement

The work in this paper for the Hawley Road Demonstration Facility was sponsored by the U.S. Environmental Protection Agency. The implementation of the findings of the Hawley Road project as applied to the Racine Root River Project were undertaken through the joint sponsorship of the U.S.E.P.A., State of Wisconsin and the City of Racine. Portions of this paper have been derived from two publications: 1) Screening/Flotation Treatment of Combined Sewer Overflows, EPA Project Report by Ecology Division, Rex Chainbelt Inc., WPCR Series 11020 FDC, January, 1972, and 2) Treatment of Combined Sewer Overflows by D.G.Mason, JWPCF, December, 1972.

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SECTION VIII

HIGH-RATE DISINFECTION OF COMBINED SEWER OVERFLOW

by

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Research Engineer
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The bacteria content of combined sewer overflow has been reported to be as high as 30 million total coliform/100 ml and 3 million fecal coliform. These levels are 1,000 to 10,000 times the allowable concentration in secondary effluents and similar restrictions have been considered for combined sewer overflows. The techniques used to remove suspended solids have in themselves no ability to remove or kill coliform. Thus bacteria kills of 3 to 4 logs (that is, 99.9% to 99.99%) are required as a separate operation for combined and separate sewer overflows.

As reported by others (1) (previous speakers) it may be possible to achieve a suitable bacteria kill with high chlorine dosages within certain types of solids removal devices so that no separate contact chamber will be required. Considerable more work needs to be done over a broad range of flow rates before the proposed advantage of dual use of this volume can be utilized on full scale plants. It is anticipated that required bacteria kills may not be obtained at low flow rates.

The special design considerations required to cope with the very high instantaneous overflow rates previously mentioned (this morning) for removal of suspended solids and organic matter hold for the disinfection equipment as well.

Conventional chlorine contact chambers installed at sewage plants are sized to provide 15 to 30 minutes detention which would require considerable area (about 1 acre per 250 acres drained at 1.0 cfs/acre). Operating close to their design rate as determined by the 2 to 1 diurnal flow variation, these basins, as often as not, fail to achieve the required bacteria kills. During the initial filling, these sewage contact chambers do not, and are not expected to, perform. A contact chamber sized to provide 15 minutes residence for a peak stormwater overflow rate would never be filled to its operating level during most storms. The operation of conventional 15-30 minute contact chambers in combined sewer overflow would be uncertain at best.

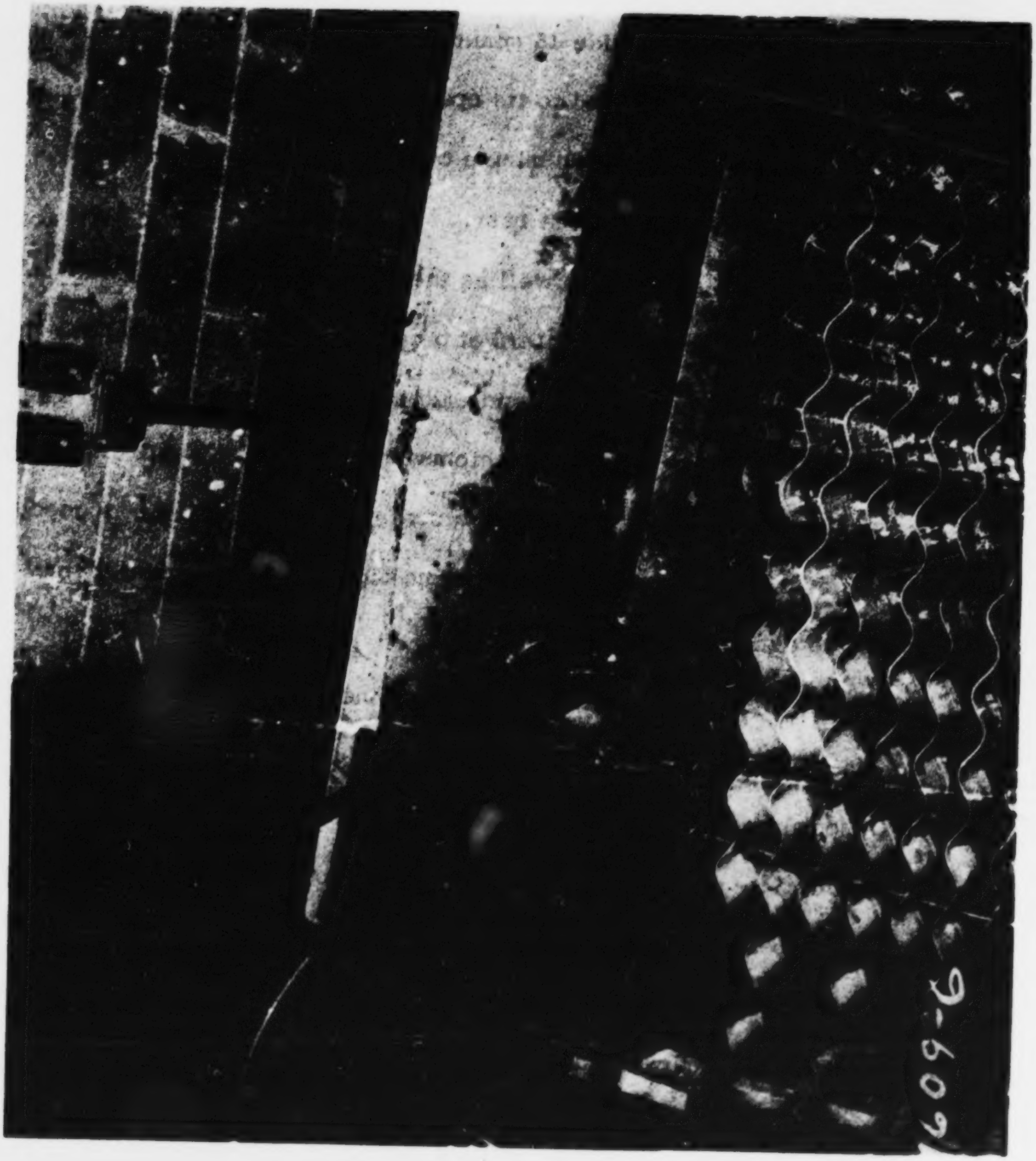
Our work on disinfection as well as the work of others (2) (3) was performed in pilot size contact chambers at a constant flow rate. That is, these chambers have not been tested at the wide (20 to 1) variations in overflow rate anticipated for a full scale chamber in stormwater service. As will be seen later, the assumption that performance of a contact chamber will be as good, if not better, at lower flow rates as it is at the higher rates is questionable even though the contact time is longer.

We have made five disinfections of combined sewer overflow while the storm was in progress. We achieved 99.99% kill (4 logs) with chlorine dosages (10 ppm) in 120 seconds. The flow rate through our units - we have two identical units - was 20 gpm. In every case, both total and fecal coliform were reduced to below 10 cells/100 ml. This performance was obtained on both the raw overflow before Microstraining and the microstrained effluent. The 3 minute chlorine demand was surprisingly uniform at about 3 ppm for the microstrained effluent and somewhat higher for the raw stormwater.

One of these chambers is shown in Figure 1. They were designed to

Figure 1

Intensely Mixed Chlorine Contact Chamber



ensure that the hypochlorite was promptly and well mixed with the stormwater. More important, or equally important, they were designed to ensure a high degree of small eddy turbulence in the passages of the contact chamber.

We attribute the extraordinarily high kill rate of these chambers to the turbulence during contact time.

The very recent literature (Collins et al (2), Kruse et al (3) and the Dow work (4)) reports several instances of laboratory studies on sewage and stormwater disinfection where similar extraordinary kill rates have been observed. Examination of the apparatus and the procedure used in these studies reveals that very high turbulence existed during these studies as well.

In one case - a beaker study by Kruse et al (3), a high stirring rate was used to demonstrate the advantage of prompt and thorough dispersion of the chlorine. Very high (4 - 5 logs) kill rate of bacteria was observed in 2 minutes when the fast stirring rate (i.e., "fast mix") condition was sustained throughout the whole study. Much poorer performance (only 1 - 2 logs in 2 minutes) was obtained at the same dosage when the more normal mixing regime of a few seconds fast mix followed by 15 minutes slow mix was used. It is of great importance that,in this study,virus were killed at high rate under the sustained fast mix condition for a few minutes whereas there was minimal virus kill even with prolonged slow mixing.

In the case of the Dow EPA (4) study, a long 1,500 ft tube was selected as a flow thru contact chamber. This configuration was apparently selected to permit precise collection of samples after a specified contact time and to avoid the problem of "true contact time vs nominal contact time" encountered in conventional flow-thru chambers due to short circuiting and back mixing, etc. The tube diameter was fortunately small enough (1-1/2" ID) relative to the flow (8 gpm) to cause turbulent flow. High kill rates were achieved with nominal chlorine dosages. For example, a 4 log kill at 2 minutes with 8 ppm dose. It is pertinent to the subject of the present paper that the Dow investigators assumed that the performance achieved in their turbulent laboratory chamber could be duplicated in a much less turbulent full scale chamber. This is, as we will see, a very questionable assumption although it is routinely made in applying laboratory data to design of full scale chambers.

Collins et al (2) also used a small diameter tube reactor in which very high velocities (13 ft/sec) caused very high turbulence. Here also, high rate kills on sewage of over 3 log in 1 minute at dosage 5 ppm were obtained. These investigators recognized, and qualitatively demonstrated (or probably the reverse order), the effect of turbulence during contact time by comparing the performance of their highly turbulent tube reactor with a gently stirred batch reactor which achieved less than 1 log kill in 1 minute at same dosage.

The basic design scheme for the pilot chambers used in our study was developed in early 1968 to support a cost estimate for 2 minute contact chambers. This cost estimate was included in the report entitled "Microstraining and Disinfection of Combined Sewer Overflows Phase I" (5).

The need for 1-2 minute disinfection equipment to complement the high rate solids removal equipment in combined sewer overflow service was apparent.

A survey of the literature to 1968 revealed no indication that 2 minute disinfection was reasonably possible.

The literature survey did reveal the situation that full scale chambers with 15-30 minutes volume routinely failed to approach the performance predicted by batch type jar tests in the laboratory. In particular, the disinfection performance of full scale chambers in cold weather was reported to be poorer than expected by temperature difference. That performance did not improve at less than design flow rate; i.e., longer contact time. These, and other considerations, suggested the parallels between disinfection rate and the flocculation rate in water treatment where a similar situation existed.

Fortunately there is a method slowly gaining acceptance by which the effect of mixing intensity on flocculation performance in laboratory and in the field can be correlated. The use of the velocity gradient (G) parameter as a measure of mixing intensity was first proposed by Camp & Stein (6) in 1943 to explain floccuation rates. They showed that velocity gradient is the difference in velocity of two parallel flowing planes of fluid in ft/sec divided by the distance between the planes in feet. Further, they showed that it was a measure of the opportunities for particle to particle (molecule) collisions per unit time per unit volume.

The product of velocity gradient times actual contact time (GT) is the number of opportunities for collision per unit volume during the flocculation operation. It follows that the GT product is proportional to the fraction of total number of particles (molecules) initially present which are actually engaged in a collision during the operation.

Several studies (7) (8) have shown that the reduction of the number of particles (i.e., the formation of a single particle from two colliding particles) is proportional to the GT product in secondary effluent flocculation. Special hardware has been developed to enhance the flocculation of sewage-like solids (9). Design and calculation methods have been developed so that the mixing intensities as measured by velocity gradient can be controlled in the laboratory (10) (11) and also reproduced in full scale equipment (12).

The application of this already developed mixing intensity technology to disinfection has been proposed by the writer (13).

The following will be a description of (a) the performance of the pilot units, (b) the preliminary design scheme, and (c) of a 92 cfs chamber designed according to this scheme.

Figure 2 shows the results of our disinfection studies to date on combined sewer overflow in an intensely mixed chlorine contact chamber. The kill is shown as the surviving fraction of,the total coliform on a log scale. Note that almost 4 logs (99.99%) are obtained with 10 ppm dosage at GT of 5,000 (2 minutes at G = 40). The contact time-mixing intensity scale is dimensionless. It is based on the nominal contact time; that is, the volume of the chamber divided by the thruput rate and is not corrected for short circuiting. The value of 9,500, for example, is the product of the G = 40 sec^{-1} velocity gradient times 240 seconds (4 minutes) nominal contact time.

For comparison, the velocity gradient in the contact chamber of a local

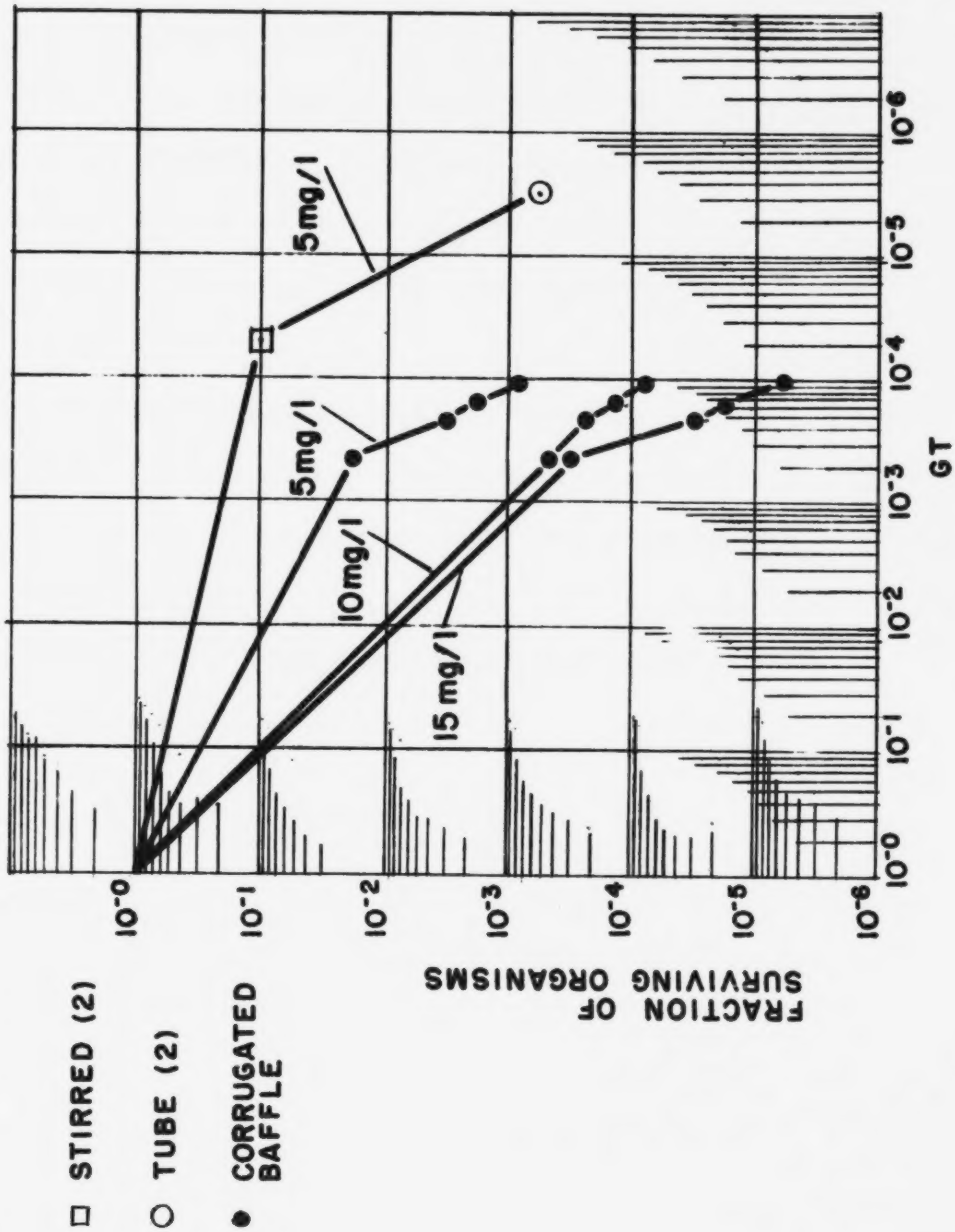

□ STIRRED (2)
○ TUBE (2)
● CORRUGATED BAFFLE
FRACTION OF SURVIVING ORGANISMS
10⁻⁰
10⁻¹
10⁻²
10⁻³
10⁻⁴
10⁻⁵
10⁻⁶
GT
10⁻⁰
10⁻¹
10⁻²
10⁻³
10⁻⁴
10⁻⁵
10⁻⁶
5mg/l
5mg/l
10mg/l
15 mg/l

FIGURE 2

sewage plant was calculated from observed velocity and head loss and found to be about 6 $\sec^{-1}$. The nominal residence time in this chamber was 1,800 seconds (30 minutes) and the GT product then was 10,000. It might be noted here that the nominal residence time is used although it has been shown (2) (14) that the true residence time is often considerably less due to short circuiting. Preliminary studies have indicated that the use of a true residence time would improve this scheme but this refinement has not been incorporated yet.

The design objective for our pilot chambers was to achieve a GT of 10,000. We arbitrarily selected 240 seconds (4 minutes) as the residence time T so that we needed a G of about 40 $\sec^{-1}$. The velocity gradient is defined (G) as:

$$G = \left[\frac{\text{Energy Dissipation Rate/Volume}}{\text{Viscosity}}\right]^{1/2}$$

For open channel flow, it has been shown (12) that:

$$G = \frac{1730}{\sqrt{\text{viscosity-cp}}}\sqrt{\text{Velocity-fps} \times \text{Channel Slope ft/ft}} \qquad \text{(Eq. 1)}$$

The viscosity is known from the lowest temperature to be considered in the design; e.g., 1.4 centipoise at 45° F.

The velocity can be arbitrarily selected at some level between 0.25 and 1.5 ft/sec, or possibly higher. The volume of the chamber has already been determined by the selected nominal residence time so that now the velocity selection also fixes the path cross-sectional area and path length.

The depth of the chamber can now be selected based upon usual considerations of soil condition, and land cost, etc., although, as will be seen later, shallower depths than usual are preferable. The remaining problem is to ensure that the required slope is obtained. The required slope is calculated from Equation 1. The slope in open channel flow can be calculated by Mannings' equation:

$$\text{Slope} = (\text{Velocity})^2 \left[\frac{n}{1.49}\right]^2 \left[\frac{1}{R}\right]^{4/3}, \qquad \text{(Eq. 2)}$$

where "n" is a factor relating to the obstruction to flow of obstacles at walls and within the channel. This factor is historically called a "roughness factor" and the numerical value found in hydraulics handbooks is 0.011 for steel or neat concrete and 0.03 for the situation where corrugated metal froms the wall of a channel whose width is several hundred times the corrugation height. For our purpose, this could be considered a turbulence promotion factor. Work is in progress to determine the effective turbulence promotion effect of corrugated baffles in narrow passages where we believe it to be at least twice the 0.03 value given above. The effect for other configurations is being studied as well. The term "roughness factor" will be used until a more appropriate term is coined.

The hydraulic radius "R" is the ratio of the cross-sectional area of the passage in ft^2 to the wetted perimeter in feet.

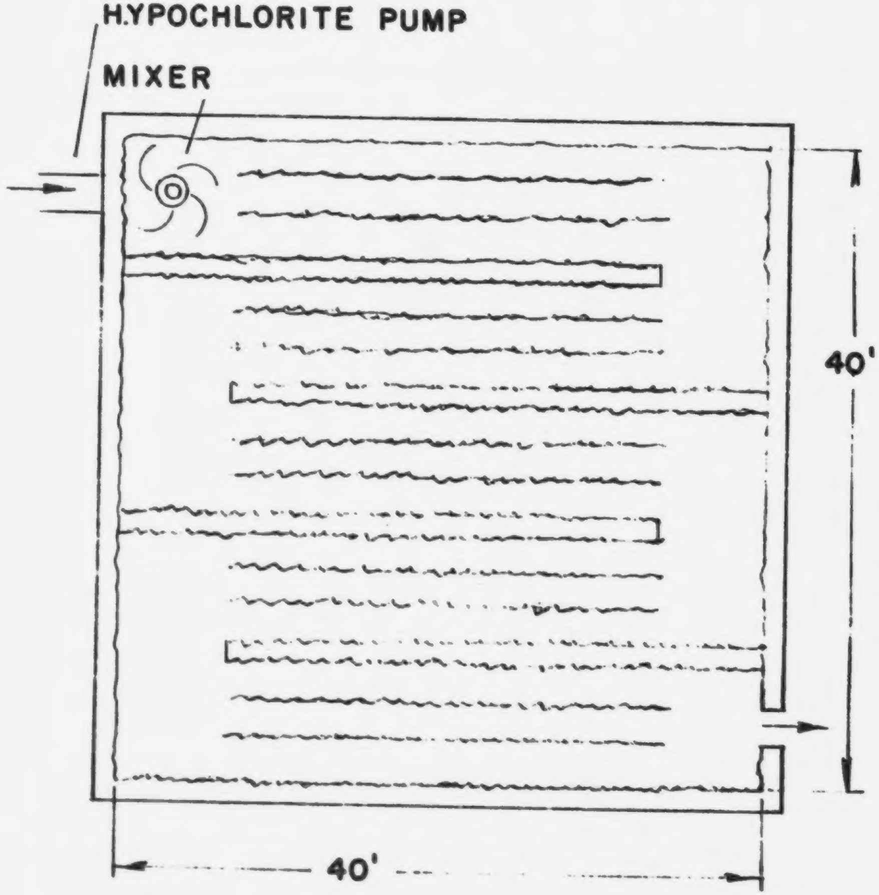
Since the velocity has been fixed and the required slope calculated, only the roughness factor relating to the type of wall and/or baffle surface and the hydraulic radius relating to the wall area parallel to the flow path can be governed by the designer.

The combined effect of these two variables is calculated from Equation 2.

For illustration in Figure 3, corrugated baffles parallel to the path are shown. In this simplified sketch, the significant dimensions are shown. The passage width is fixed by the selected velocity and channel depth. The number of the parallel baffles inserted determines the hydraulic radius. The roughness factor is determined primarily by the surface of the baffle material selected.

In spite of the undeveloped state of this design scheme, we were able to produce a chamber within 6% (9,400) of the design target (10,000) on our first attempt. Also, additional baffles can be easily inserted at a later date if required.

This design scheme yields considerable insight to the evaluation of the performance of existing and future contact chambers. The disinfection performance has been shown to be a function of the GT parameter. In conventional chambers the outlet weir is located near the design rate water level so that the water volume is nearly constant at all flow rates. As can be seen by Equations 1 and 2, the G varies as the $(\text{velocity})^{1.5}$. With constant liquid level, the T varies as (1/velocity), thus the GT parameter will vary as the $(\text{velocity})^{.5}$ or with $(\text{flow rate})^{.5}$. This poorer performance at reduced flow rate would escape attention under relatively constant rate conditions in a sewage plant. However, under the widely variable rate conditions met in


H.YPOCHLORITE PUMP
MIXER
40'
40'

Reproduced from best available copy.

TOP VIEW OF A 92 CFS (60 MGD)
INTENSLY MIXED CHLORINE CONTACT CHAMBER
(AVG. WATER DEPTH 7') RESIDENCE TIME 120 SECONDS

FIGURE 3

combined sewer overflow service, it must be considered. The use of a Sutro weir has been proposed to maintain a constant velocity at all flow rates.

A 92 cfs (60 mgd) Intensity Mixed Chlorine Contact Chamber has been designed. This chamber was designed to follow a microstrainer facility with 46 cfs treatment capacity and an additional 46 cfs bypass capacity. The chlorine contact chamber was designed to have 120 seconds residence time at the 92 cfs rate and, since a Sutro weir is used the residence time at less than the 92 cfs rate will be about 120 seconds also. The velocity is 1.5 ft/sec and the amount of baffling and its configuration is such to yield a velocity gradient G of 40, as in our pilot plant.

The chamber is 40' by 40' and has an average liquid depth of 7' at maximum flow. Internal walls form a labyrinthine-like passage of 8' in width and produce a velocity of 1.5 ft/sec. The internal walls are faced with a commercially available corrugated asbestos siding having 1-1/2" deep corrugations.

Two additional corrugated panels are mounted as parallel baffles in the channels forming 32 inch wide passages. The baffles extend from liquid level to within a foot of the floor. Ideally the floor would be similiarly corrugated, but this is not necessary. The head loss through the chamber at peak flow is about 8 inches. (See Figure 3)

The inlet to the chamber is equipped with a 3 hp mixer sweeping an 8' x 8' section of the channel (about 5 sec residence time). A mixer of this horsepower should be able to impart 1 hydraulic horsepower to the water to enduce a mixing intensity of about 200 sec^{-1} in this 450 ft^3 volume, which should be adequate for thoroughly mixing the chlorine chemicals. Such a provision for mixing of chemicals is incomparably superior to the methods usually used in sewage plants. The mixer should be of such a type that it can operate at varying water levels from 7' down to 1'.

The outlet of the chamber should be fitted with a relatively narrow outlet weir placed as low as the available outfall head will allow, preferably at the bottom. Further, the outlet weir should be of the Sutro type to maintain the velocity in the chamber, at less than peak rate, as near the peak rate velocity as possible. A Sutro weir at the bottom will maintain peak rate velocity at all flow-thru rates. In the event the allowable outfall head will not permit placing the weir at the bottom, a small pump must be provided to empty the chamber at the end of the storm.

The installed cost of such a chamber has been calculated to be about \$53,000 (in 1969 dollars) less the cost of land, engineering and profit (1). It is difficult to compare costs developed by different estimators. However, this cost can be compared to the data developed by Smith (15) of \$25,000 for an \$11,000 ft^3 basin, which is the volume of the basin described above. Also, it can be compared to Smith's estimate of \$90,000 for the 81,000 ft^3 chamber required to provide 15 minutes residence for 60 mgd in a conventional chamber.

The inherent advantage of increased turbulence economically induced in this type of installation to enhance reaction rates can be used in many situations. An obvious example would be to use it in chlorine contact chambers at sewage plants with savings in construction cost, land, and the advantage of high virus kill and reliable bacteria kill.

ACKNOWLEDGEMENT

This work was conducted with the City of Philadelphia in two phases, (1) under a contract from the Environmental Protection Agency to the Cochrane Division of the Crane Co., and (2) under an EPA grant to the City of Philadelphia, The efforts of City personnel were under the general direction of Carmen Guarino, Water Commissioner, with William Wankoff and M. Lazanoff, serving as Project Director and Laboratory Director. J. Radzuil headed the City's R and D Department who also lent valuable assistance.

The assistance and guidance of these people are gratefully acknowledged.

The overall guidance and helpful advice of Richard Field, Project Officer, EPA, Edision, New Jersey, were most valuable.

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SECTION IX

THE SWIRL CONCENTRATOR AS A COMBINED SEWER OVERFLOW REGULATOR

by

Richard H. Sullivan
Assistant Executive Director
American Public Works Association

A report by the American Public Works Association published in 1970 gave the results of a study of combined sewer overflow regulator facilities. Design, performance and operation and maintenance experiences from the United States and Canada, and in selected foreign countries were reported. It was evident that North American practice has emphasized the design of regulators simply as flow splitters, dividing the quantity of combined sewage to be directed to the treatment facitilities, and the overflow to receiving waters. Little consideration was given to improving the quality of the overflow wastewater.

Using hydraulic laboratory tests and mathematical modeling strongly we have determined that it is possible to remove significant portions of settleable and floatable solids from combined sewage overflows by using a swirl concentrator. The practical, simple structure has the advantages of low capital cost; absence of primary mechanical parts should reduce maintenance problems; and construction largely with inert material should minimize corrosion. Operation of the facility is automatically induced by the inflowing combined sewage so that operating problems normal to dynamic regulators such as clogging will be very infrequent.

The device, as developed, consists of a circular channel in which rotary motion of the sewage is induced by the kinetic energy of the sewage entering the chamber. Flow to the treatment plant is deflected and discharges through an orifice called the foul sewer outlet, located at the bottom and near the center of the chamber. Excess flow in storm periods discharges over a circular weir around the center of the tank and is conveyed to storage treatment devices as required or to receiving waters. The concept is that the rotary motion causes the sewage to follow along a spiral path through the circular chamber.

A free surface vortex was eliminated by using a flow deflector, preventing flow completing its first revolution in the chamber from merging with inlet flow. Some rotational movement remains, but in the form of a gentle swirl, so that water entering the chamber from the inlet pipe is slowed down and diffused with very little turbulence. The particles entering the basin spread over the full cross section of the channel and settle rapidly. Solids are entrained along the bottom, around the chamber, and are concentrated at the foul sewer outlet.

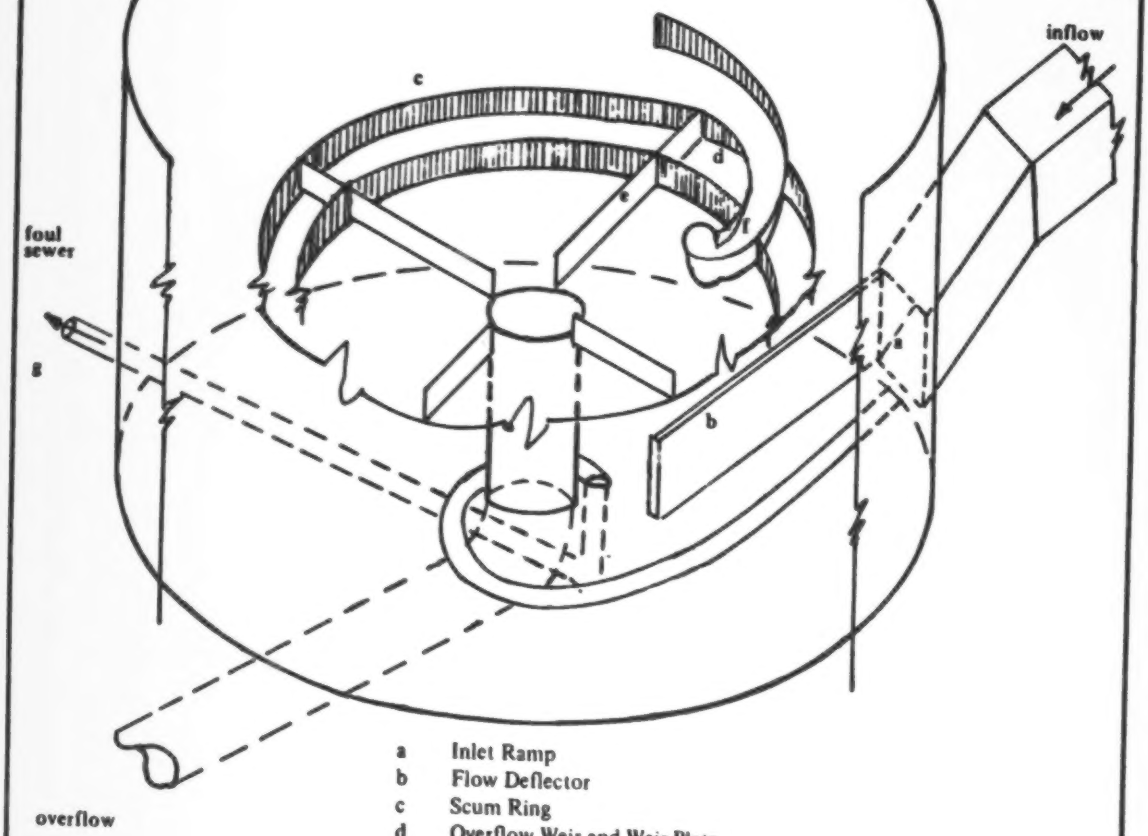
Figure 1, Isometric View of Swirl Concentrator, depicts the final hydraulic model layout showing details such as the floatables trap, foul outlet and floor gutters.

The swirl concentrator may have practical applications as a degritter, or grit removal device for sanitary sewage flows or separate storm water discharges of urban runoff waters. It may have capabilities for the clarification of sanitary sewage in treatment plants, in the form of primary settling or, possibly, final settling chambers. It might be used for concentrating, thickening, or elutriating sewage sludges. It may be serviceable in the separation, concentration and recycling of certain industrial waste waters, such as pulp and paper wastes or food processing wastes, with reuse of concentrated solids and recirculation of clarified overflow waters in industrial processing closed circuit systems.

In water purification practices, it may find feasible applications in chemical mixing, coagulation and clarification of raw water. Other uses may prove to be realistic and workable.

Complete reports describing the hydraulic laboratory study and the mathematical modeling are included in the report EPA R2-72-008, September 1972, published by USEPA. The body of the report details the basis of the assumptions used to establish the character and amount of flow to be treated and the design of a swirl concentrator based upon the hydraulic and mathematical studies.

Although the study was performed for the City of Lancaster, Pennsylvania, with a specific point of application defined, all work was accomplished in a manner which allows ready translation


inflow
c
d
e
f
a
b
foul sewer
g
overflow

a	Inlet Ramp
b	Flow Deflector
c	Scum Ring
d	Overflow Weir and Weir Plate
e	Spoilers
f	Floatables Trap
g	Foul Sewer Outlet
h	Floor Gutters

FIGURE 1
ISOMETRIC VIEW OF SWIRL CONCENTRATOR

application of the results to conditions which might be found at other installations and for other purposes.

Consideration of the use of a swirl concentrator as a combined sewer overflow regulator facility requires an evaluation of many factors which include:

1. hydraulic head differential between the collector and interceptor sewers and head available in collector sewer to allow insystem storage;
2. hydraulic capacity of collector sewer;
3. design flow;
4. dry-weather flow and capacity of interceptor sewer; and
5. amount and character of settleable solids.

Although many of these items have been mentioned in the preceding sections of the report, the importance of each will be highlighted in order to emphasize the importance of each point in a preliminary evaluation of the use of the swirl concentrator.

Hydraulic Head Differential. There must be sufficient hydraulic head available to allow dry-weather flows to pass through the facility and remain in the channel. The total head required for operation is shown in Figure 2, Hydraulic Head Requirements. Determination of the maximum elevation in the collector sewer that can be utilized for insystem storage and the differential elevation between the collector and interceptor sewers is the total available head.

The head required will vary directly with flow and the outlet losses in the foul sewer.

If sufficient head is not available to operate the foul sewer discharge by gravity, an economic evaluation would be necessary to determine the value of either pumping the foul sewer outflow continuously, or pumping the foul flow during storm conditions and bypassing the swirl concentrator during dry-weather conditions, perhaps with a fluidic regulator.

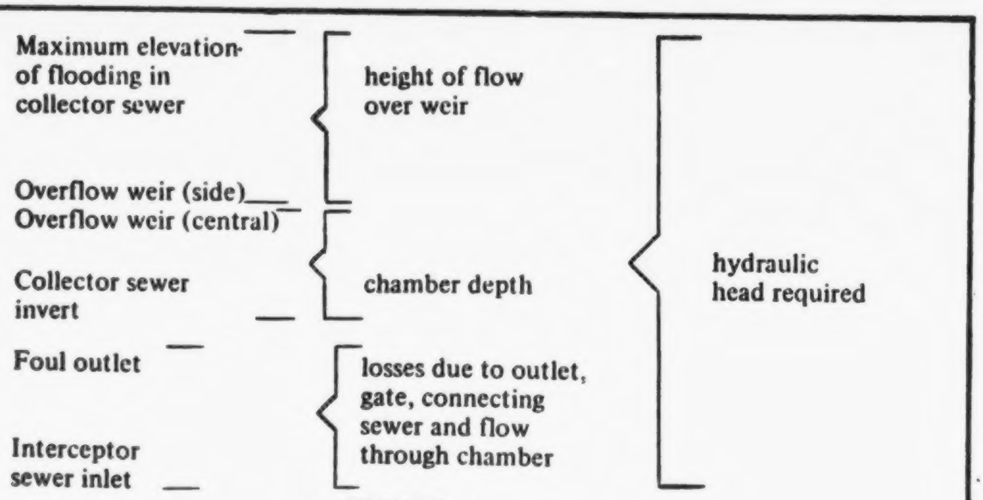

Maximum elevation of flooding in collector sewer
Overflow weir (side)
Overflow weir (central)
Collector sewer invert
Foul outlet
Interceptor sewer inlet
height of flow over weir
chamber depth
losses due to outlet, gate, connecting sewer and flow through chamber
hydraulic head required

FIGURE 2
HYDRAULIC HEAD REQUIREMENTS

Hydraulic Capacity of Collector Sewer System. The facility must be designed to handle the total flow which might be delivered by the collector system. Thus a study of the drainage area must be made to determine the limiting grade and pipe sizes which control the quantity of flow. Solids removal from a peak flowrate may not be required. If the chamber is not designed for such maximum flows, however, velocity energies which could be developed at such full flow conditions should be avoided by providing a bypass in the form of a side overflow weir.

Design Flow. Selection of the design flow for sizing the chamber should be accomplished on the basis of a complete hydrological study to determine frequency and amount of precipitation which can be anticipated as well as runoff hydrographs. Computer models such as developed by the University of Florida for USEPA can be of assistance in determining the solids load which may be associated with various amounts and intensity of precipitation. Provision of maximum solids removal for a two-year frequency storm for the Lancaster, Pennsylvania, project was made on the basis of engineering judgment and an evaluation of local receiving water conditions. As the cost of construction will increase in direct proportion to design flow, an economical evaluation should generally be used to select the flow capacity. The efficiency curve for the facility is rather flat over a wide range of flows, resulting in perhaps large increases in cost for marginal improvements in efficiency.

A major constraint in selecting large design flows is the anticipated shoaling problems of solids at low flow rates in large facilities. Self cleaning is enhanced by reduced diameters. This consideration may make it desirable to design for lower flows, particularly where some form of overflow treatment is to be provided. Again the computer model can be used to determine the magnitude of the solids carry-over problem to the secondary device.

A third consideration is the maintenance of low-inflow velocities, with turbulence minimized. At the design flow the inflow velocity should be in the range of three to five fps. The inflow velocity may require reduction by enlarged pipe sections or other means to achieve this rate.

Dry Weather Flow and Capacity of Interceptor Sewer. Sizing of the foul sewer, the foul outlet and the gutter depend upon a determination of the dry-weather flow in addition, the capacity of the interceptor sewer to handle the foul flow must be known. The foul sewer must be large enough to maintain and not be subject to blockage---usually a minimum 12-inch diamter. However, the head on the outlet during overflow conditions will allow considerable variations in the foul discharge if it is not controlled.

The efficiency of the chamber is affected by the ratio of foul flow to overflow---although there appears to be a broad operating range over which reasonable removal efficiencies can be maintained.

Maximum advantage should be taken of capacity in the interceptor system, particularly during the period when the chamber is being drawn down. Thus, sensing of the flow in the interceptor and the use of a control gate on the foul sewer appear desirable to obtain maximum results from the use of the chamber.

Amount of Character of Settleable Solids. The sewer system must provide capacity to handle the increase in settleable solids which will be captured from the combined sewer overflow and discharged to the treatment plant. In the case of Lancaster, Pennsylvania, this could amount to more than a ton of solids from one device in a very short period of time. Additional grit removal and sludge processing equipment may be necessary. Should the foul flow be pumped, sumps and pumps should be designed to handle the anticipated high solids content.

If the settleable solids which can be anticipated in the combined sewer overflow can be defined by the amount, specific gravity, and particle size, the mathematical and the hydraulic model may be used to determine the size of the chamber required to achieve desired levels of solids removal. Ordinarily this will not be feasible and the flow criteria developed by the hydraulic model will be used to design the facility and predict removal efficiencies.

In order to evaluate the efficiency of the chamber, facilities should be provided for sampling the inflow, foul sewer flow and overflow. Settleable solids should be delineated in all of these flows. The quantity of inflow and foul sewer flow should also be measured. Difficulties in obtaining representative samples from any of the flows may make evaluation difficult. However, the treatment plant or combined sewer overflow treatment facility , if used, should provide an excellent means of making a gross evaluation into the effectiveness of the chamber.

Provision of a means to measure the depth of flow over the weir should act to give a reliable measurement of the flow when added to the quantity of flow to the foul sewer.

Data from many full-scale operations, operating with various flow conditions and solid loadings will be necessary to properly evaluate the usefulness of the swirl concentrator as a combined sewer overflow regulator.

Cost of Facility. The cost of construction of the swirl concentrator will vary with the length of inlet pipe which must be reconstructed, the depth of the chamber and the nature of the material to be excavated, the need for a roof, and the general site conditions under which the work will be conducted. The materials of construction will usually be concrete and steel and elaborate form work will not be required.

For the Lancaster, Pennsylvania, application where a 36 foot diameter chamber in limestone is contemplated, the preliminary estimate of cost was $100,000 in 1972 costs. This cost estimate included a roof, foul sewer outlet control and a wash-down system. Site construction problems are minimized in as much as the construction will be off of the street right-of-way.

SECTION X

THE EPA STORMWATER MANAGEMENT MODEL

A CURRENT OVERVIEW

by

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University of Florida
Gainesville, Florida 32611

1. INTRODUCTION

A. COMBINED AND STORM SEWER OVERFLOWS

An enormous pollution load is placed on streams and other receiving waters by combined and separate storm sewer overflows. It has been estimated that the total pounds of pollutants (BOD and suspended solids) contributed yearly to receiving waters by such overflows is of the same order of magnitude as that released by all secondary sewage treatment facilities (Gameson and Davidson, 1964; Field and Struzeski, 1972). The Environmental Protection Agency (EPA) has recognized this problem and led and coordinated efforts to develop and demonstrate pollution abatement procedures (Field and Struzeski, 1972). These procedures include not only improved treatment and storage facilities, but also possibilities for upstream abatement alternatives such as rooftops and parking lot retention, increased infiltration, improved street sweeping, retention basins and catchbasin cleaning or removal. The complexities and costs of proposed abatement procedures require that care and effort be expended by municipalities and others charged with decision making for the solution of these problems.

B. THE STORM WATER MANAGEMENT MODEL

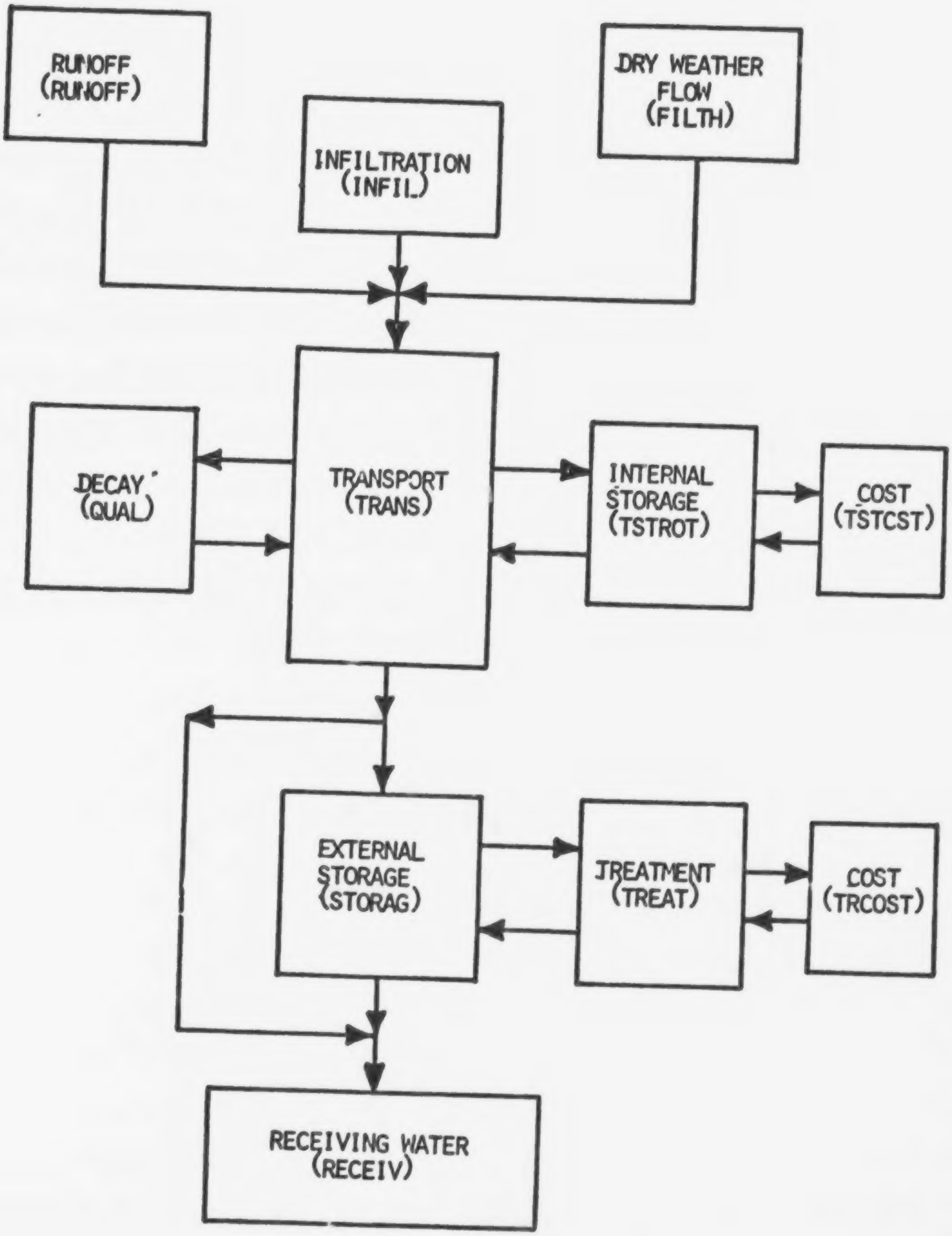
It was recognized that an invaluable tool to decision makers would be a comprehensive mathematical computer simulation program that would accurately model quantity (flow) and quality (concentrations) during the total urban rainfall-runoff process. This model would not only provide an accurate representation of the physical system, but also provide an opportunity to determine the effect of proposed pollution abatement procedures. Alternatives could then be tested on the model and least cost solutions could be developed.

As a result, the University of Florida (UF), Metcalf and Eddy, Inc., Engineers (ME) and Water Resources Engineers (WRE) were awarded a joint contract for the development, demonstration and verification of the Storm Water Management Model (SWMM). The resulting model, completed in October, 1970, has been documented (EPA, 1971a, b, c, d) and is presently being used by a variety of consulting firms and universities.

The present SWMM is descriptive in nature and will model most urban configurations encompassing rainfall, runoff, drainage, storage-treatment, and receiving waters. The major components of the SWMM are illustrated in Figure 1-1. However, it does not define nor determine any decisions for the system or consider alternative methods for efficient economic comparisons.

C. DECISION MAKING

In recognition of the need for improved decision making capabilities, the University of Florida submitted a proposal to EPA titled "A Decision Making Model for the Management of Storm Water Pollution Control" in which it was intended to provide a systematic procedure which could be applied to a wide variety of specific circumstances in support of intelligent management decisions. The work required to obtain a least cost solution would be considerably


RUNOFF
(RUNOFF)
INFILTRATION
(INFIL)
DRY WEATHER
FLOW
(FILTH)
DECAY
(QUAL)
TRANSPORT
(TRANS)
INTERNAL
STORAGE
(TSTROT)
COST
(TSTCST)
EXTERNAL
STORAGE
(STORAG)
TREATMENT
(TREAT)
COST
(TRCOST)
RECEIVING WATER
(RECEIV)

Note: Subroutine names are shown in parentheses.

Figure 1-1
Overview of Model Structure

reduced by means of determining the origin of the most severe pollution load, consideration of all upstream and downstream pollution abatement procedures and associated costs, and through the possible use of mathematical optimization techniques.

The project was funded as part of an EPA Demonstration Grant to Lancaster, Pennsylvania (Federal Grant No. 11023GSC), in which an underground "silo," a swirl concentrator and a microstrainer were to be installed at the outfall of the Stevens Avenue Drainage District to control overflow into the Conestoga Creek (details are presented in the next section).

Results of the decision-making methodology and other aspects of the research have recently been formulated (Heaney and Huber, 1973). Decision-making for urban storm water management is presented in the broader context of urban water resources management. Pollution sources and control options are inventoried and accompanied by economic data. Performance standards are considered and the importance of automobile-related facilities (*e.g.*, streets, parking lots, curbs and gutters) as contributors to storm water pollution and quantity is emphasized. Finally, a linear programming and game theory approach is used to develop efficient and equitable control strategies.

This paper presents an overview of the SWMM by illustrating its use in Lancaster; the following section is taken from the Final Report (Heaney and Huber, 1973) from which other details are available. Major revisions to the Model have been made to include urban erosion prediction, modeling of new treatment devices and biological treatment facilities, monitoring of significant pollution sources, flexibility in modeling new areas, new and improved cost functions for treatment and storage options and a modest hydraulic design capability as well as minor programming changes and slight format revisions. The SWMM has proven to be a useful and economical tool in the assessment of urban storm water problems. Individual runs described in the following section, for instance, could be accomplished using less than three minutes of CPU time on the IBM 370/165 at the University of Florida Computing Center, for a Runoff-Transport-Storage/Treatment-Receiving simulation. Although computational changes vary, they are well within reasonable bounds.

2. TESTING IN LANCASTER, PENNSYLVANIA

The City of Lancaster, Pennsylvania, population 79,500, is situated in a drainage area of about 8.24 square miles (5,274 acres). The receiving stream in the Lancaster area is the Conestoga Creek which drains an area of approximately 473 square miles into the Susquehanna River. The average flow is 387 cubic feet per second with a maximum recorded flow of 22,800 cubic feet per second.

There are two sewage treatment plants within the city, both of which discharge into the Conestoga Creek. The North Plant with a capacity of 10 mgd serves a population of 36,000 people, and the South Plant recently expanded from 6 mgd to 12 mgd and is designed to serve 69,000 people. Both plants provide secondary treatment. About one third of the flow to the North Plant is derived from areas with separate sewers outside the city serving an estimated population of 17,500 people and some industries. The remaining two thirds of the sewage flow to the North Plant is derived from the combined sewers serving the north part of the city plus about 250 suburban acres estimated to have 18,500 people and many water-using industries. In addition, most of the year the water table is high resulting in considerable infiltration. An overflow line diverts excess flow to the Conestoga during wet weather. The North Plant drainage area is estimated at 3.72 square miles.

The South Plant is designed to handle a population of 34,500 served by combined sewers and, in addition, up to an approximately equal amount from separated sewers throughout the surrounding area. The South Plant drainage area encompasses 4.52 square miles and is comprised of four districts. Stevens Avenue district which is the subject of EPA demonstration grant is one of the four districts connected to the South Plant. Three of the districts, including Stevens Avenue, pump the sewage from a receiving station within the district to the South Plant. All locations have overflow arrangements that discharge into the Conestoga Creek when the capacity of the system is exceeded.

The total drainage area of the Stevens Avenue district is 227 acres which, while only about 4.3% of the total Lancaster drainage area served by North and South treatment plants, is 17% of the drainage area designed to flow into the South Plant from combined sewers. The population within the Stevens Avenue district is estimated at 3,900. Figure 2-1 illustrates various drainage districts within the city.

1. DEMONSTRATION GRANT DESCRIPTION

In order to remedy the situation resulting from combined sewer overflows, the City of Lancaster decided to explore means other than sewer separation. Construction of several underground silos at various locations within the city is contemplated for retention of overflow during wet periods and subsequent pumping to the treatment plants during low flow periods.

Stevens Avenue district was selected as the demonstration site for evaluation of the effectiveness of a silo in combating combined

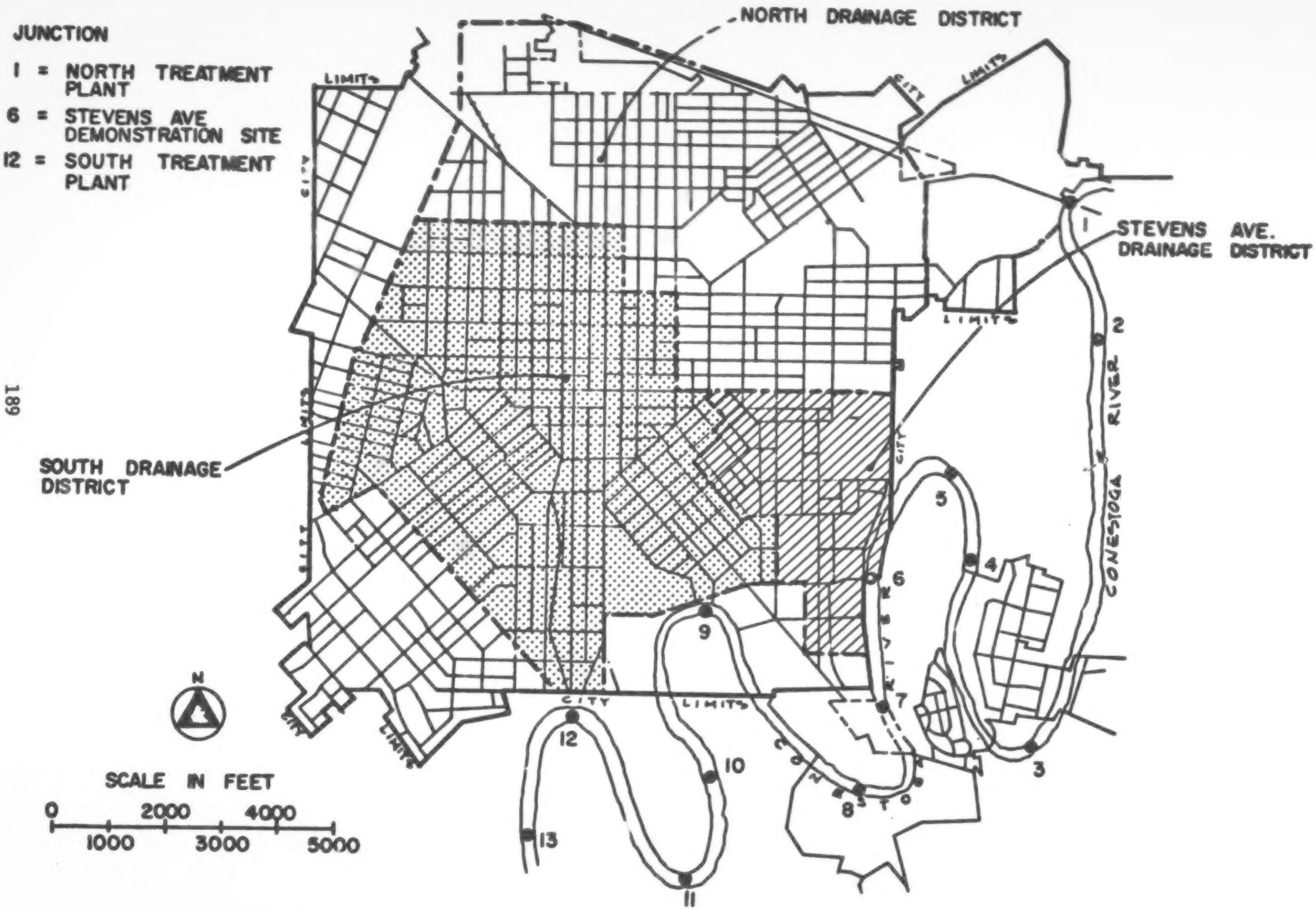

JUNCTION
1 = NORTH TREATMENT PLANT
6 = STEVENS AVE DEMONSTRATION SITE
12 = SOUTH TREATMENT PLANT
NORTH DRAINAGE DISTRICT
STEVENS AVE. DRAINAGE DISTRICT
SOUTH DRAINAGE DISTRICT
CITY LIMITS
CONESTOGA RIVER
1
2
3
4
5
6
7
8
9
10
11
12
13
N
SCALE IN FEET
0
1000
2000
3000
4000
5000

Figure 2-1
Drainage Districts of Lancaster, Pennsylvania and Numbering System for Receiving Junctions.

sewer overflows. The sewer layout for Stevens Avenue district is shown in Figure 2-2. During normal dry weather periods, the dry weather flow is pumped to the South treatment plant. During wet periods, when the incoming flow to the pump station exceeds the capacity of the station, the overflow discharges directly into the Conestoga Creek through a 60 inch sewer located at point 6 on Figure 2-1.

The City of Lancaster also authorized APWA to develop design parameters for a full-scale swirl concentrator for removal of solids prior to the retention of flow in the underground silo. Location of the demonstration site is shown in Figure 2-2. A flow diagram of the proposed swirl concentrator-silo treatment is presented in Figure 2-3. In order to fully evaluate this treatment the city decided to include chlorination and microstraining as a part of this demonstration project. The capacity of the silo is expected to be 160,000 cf.

The tasks assigned to the University of Florida were as follows:

1) Conduct further verification and testing of the Storm Water Management Model based on active overflow measurements on selected storm events and to make refinements to the Model;

2) Provide results of simulations to the APWA in order for it to develop design criteria and sizing of the swirl concentrator;

3) Simulate the effect of the swirl concentrator-underground silo treatment; and

4) Simulate the effect of combined sewer overflow from the entire city to the Conestoga Creek.

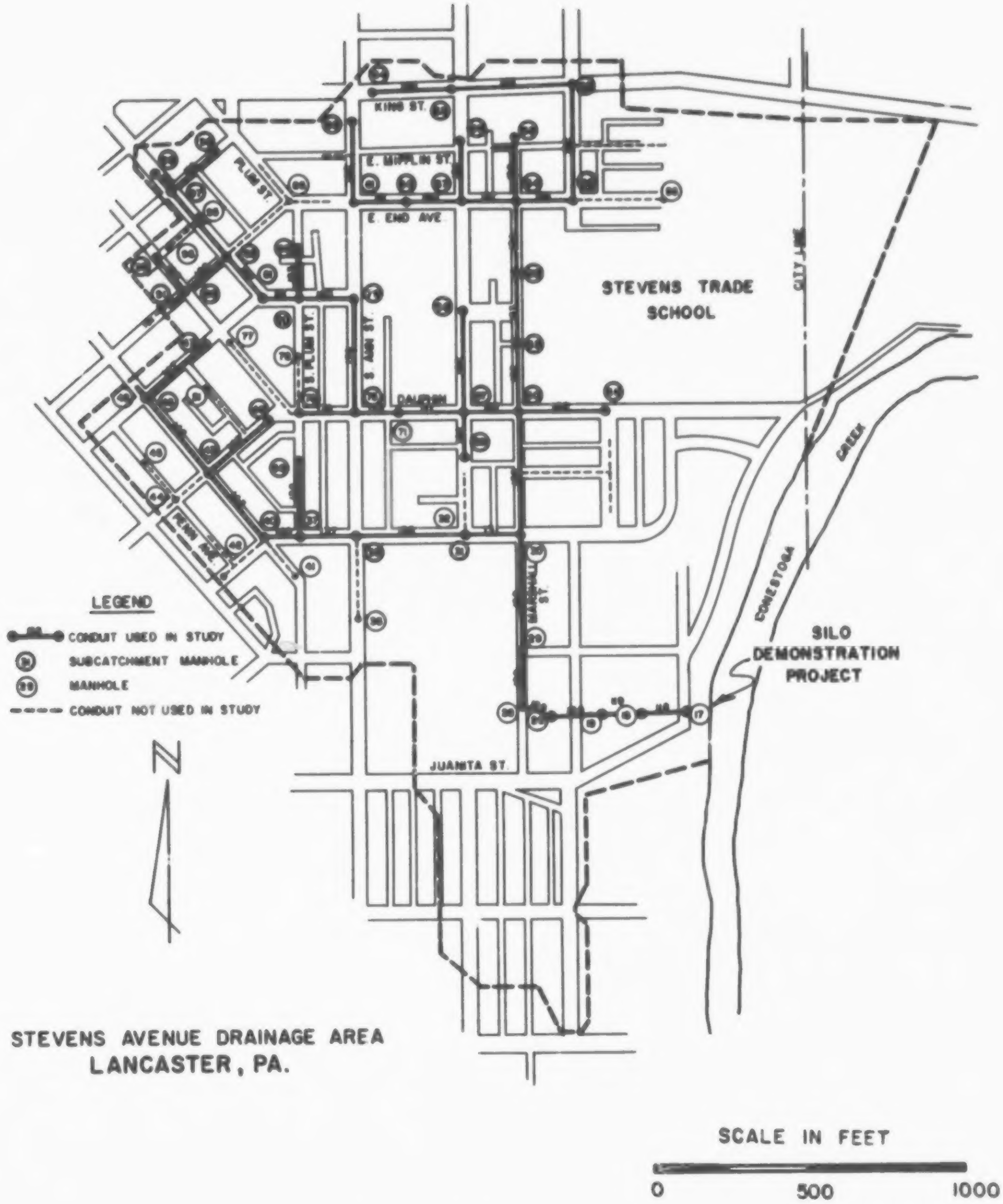
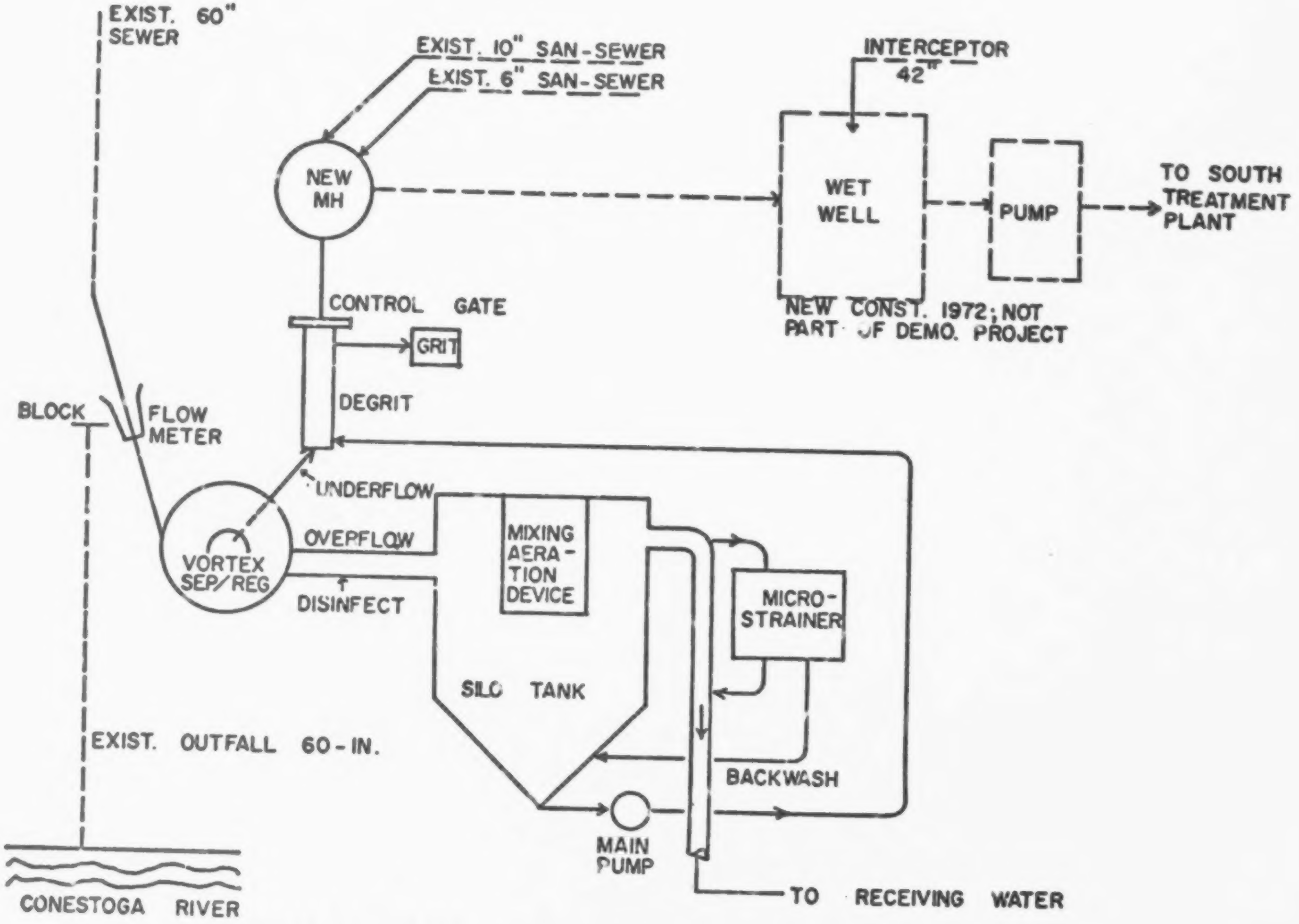

KING ST.
E. MIFFLIN ST.
PLUM ST.
E. END AVE.
STEVENS TRADE SCHOOL
CITY LINE
S. PLUM ST.
S. ANN ST.
DAUPHIN
PENN AVE.
CONESTOGA
CREEK
SILO DEMONSTRATION PROJECT
JUANITA ST.
LEGEND
CONDUIT USED IN STUDY
SUBCATCHMENT MANHOLE
MANHOLE
CONDUIT NOT USED IN STUDY
N
STEVENS AVENUE DRAINAGE AREA
LANCASTER, PA.
SCALE IN FEET
0
500
1000

Figure 2-2
Stevens Avenue Drainage Area with Runoff-Transport Numbering System.


EXIST. 60" SEWER
EXIST. 10" SAN-SEWER
EXIST. 6" SAN-SEWER
INTERCEPTOR 42"
NEW MH
WET WELL
PUMP
TO SOUTH TREATMENT PLANT
NEW CONST. 1972; NOT PART OF DEMO. PROJECT
CONTROL GATE
GRIT
DEGRIT
BLOCK
FLOW METER
UNDERFLOW
OVERFLOW
DISINFECT
VORTEX SEP/REG
MIXING AERATION DEVICE
MICRO-STRAINER
SILO TANK
EXIST. OUTFALL 60-IN.
BACKWASH
MAIN PUMP
CONESTOGA RIVER
TO RECEIVING WATER

PRELIMINARY LANCASTER FLOW DIAGRAM

Figure 2-3
Flow of Treatment-Storage Options at Demonstration Site.

2. DESCRIPTION OF THE STEVENS AVENUE RUNS

A total of four studies comprising nine storms were simulated. The city and its engineers provided input data as well as two overall measurements. The Stevens Avenue district was subdivided into 41 subcatchments. A description of each study and its results are given below:

Study No. 1.--The first study was based on a series of storms between July 29 and August 3, 1971. This six-day period deposited a record amount of precipitation throughout the Lancaster area (variously measured between 7.3 and 9.46 inches). During four of the six days, the storms were very intense over short periods; in one case, being the second heaviest of record. For purposes of simulation, Study No. 1 was divided into six storms. The amount and times of precipitation assumed for each of these six storms are shown in Figures 2-4 through 2-9 and results of computer simulations for each of these storms are shown in the same figures. These figures show the expected quantity and quality of the overflow from the Stevens Avenue district for a given rainfall. These runs indicate that an overflow as high as 400 cfs may be expected for a storm event similar to Storm No. 6.

These computer runs also indicate that total suspended solids and BOD discharges expected in the overflow may be on the order of magnitude of 778 pounds and 635 pounds respectively for Storm No. 5 and 849 pounds and 768 pounds respectively for Storm No. 6. Unfortunately, since actual flow measurements were not taken during this study, it was not possible to determine the actual overflow quantity and quality. However,

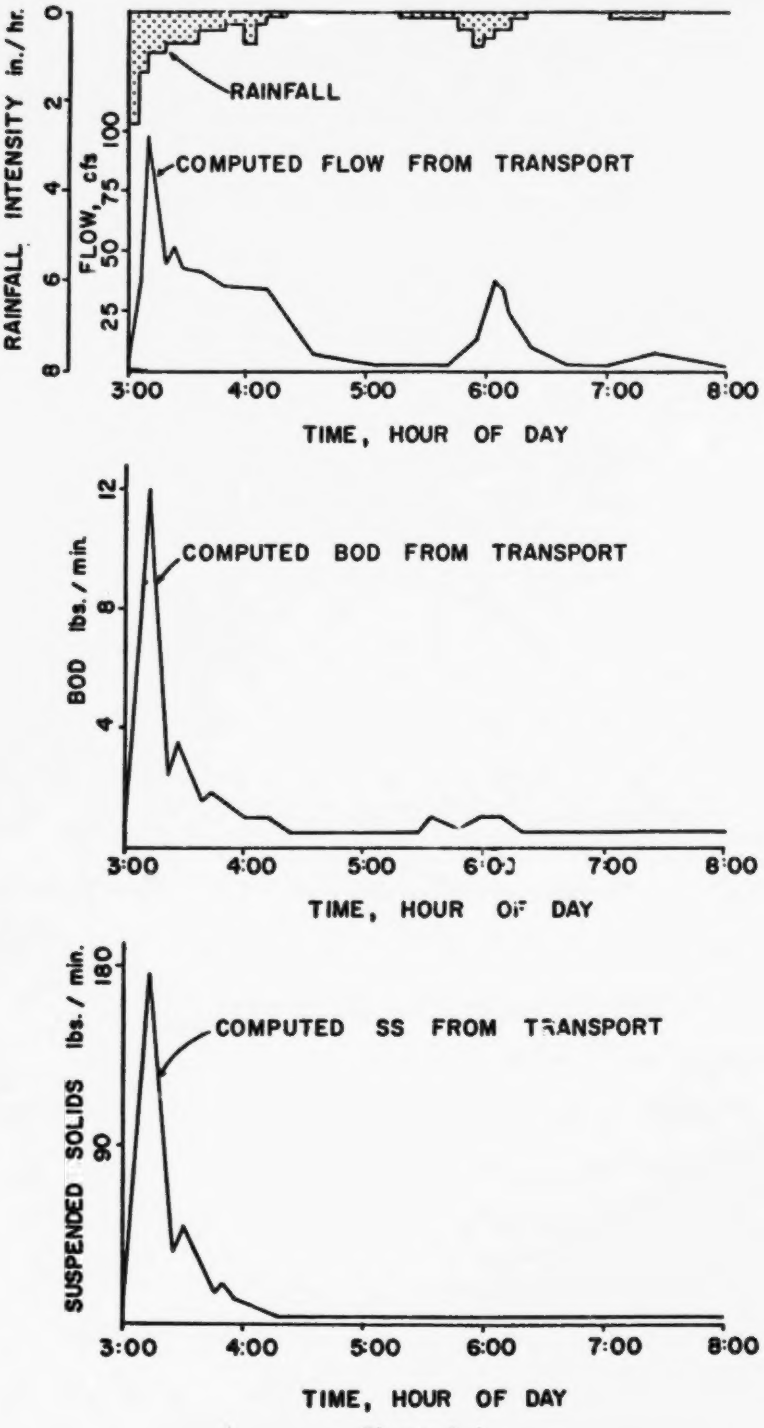


Figure 2-4
Runoff-Transport Simulation for Stevens Avenue.
Study 1, Storm 1.

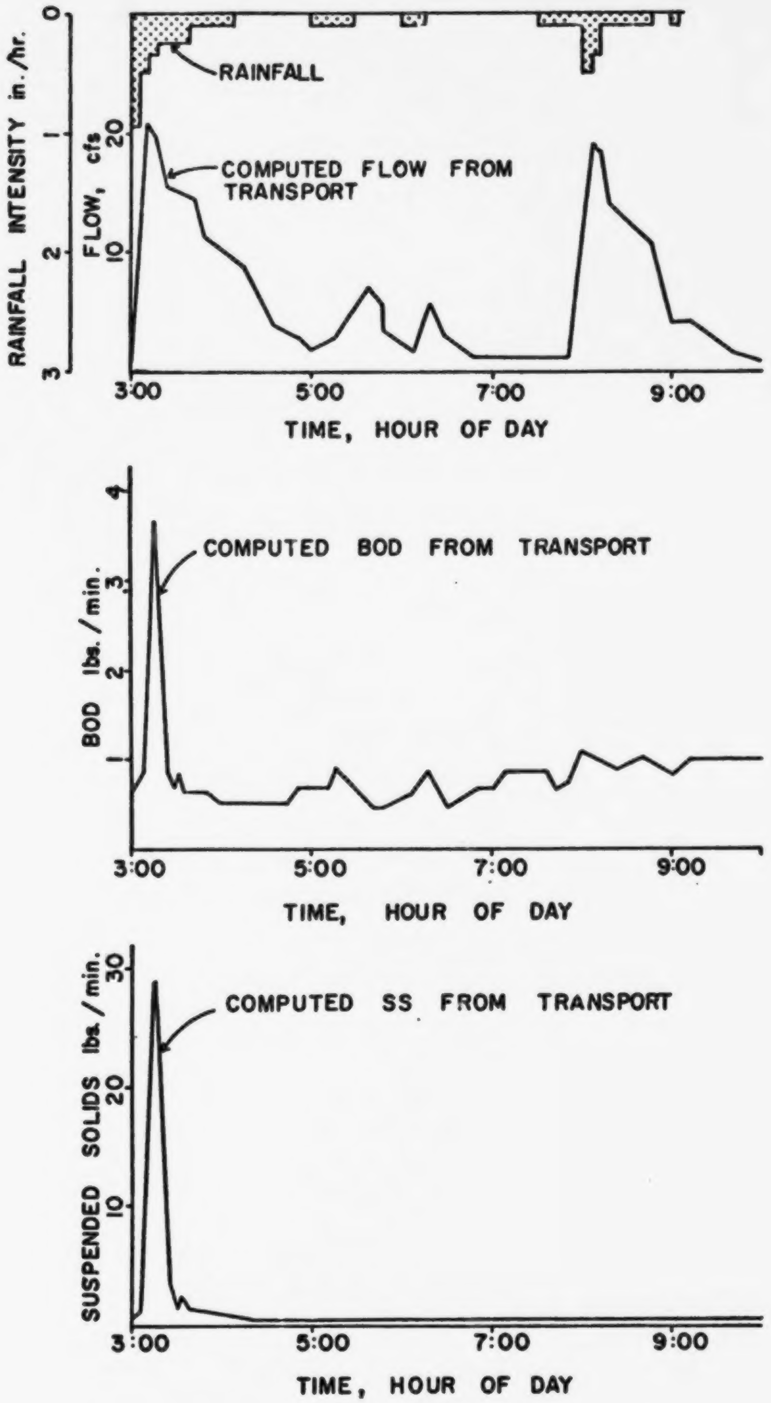


Figure 2-5
Runoff-Transport Simulation for Stevens Avenue.
Study 1, Storm 2.

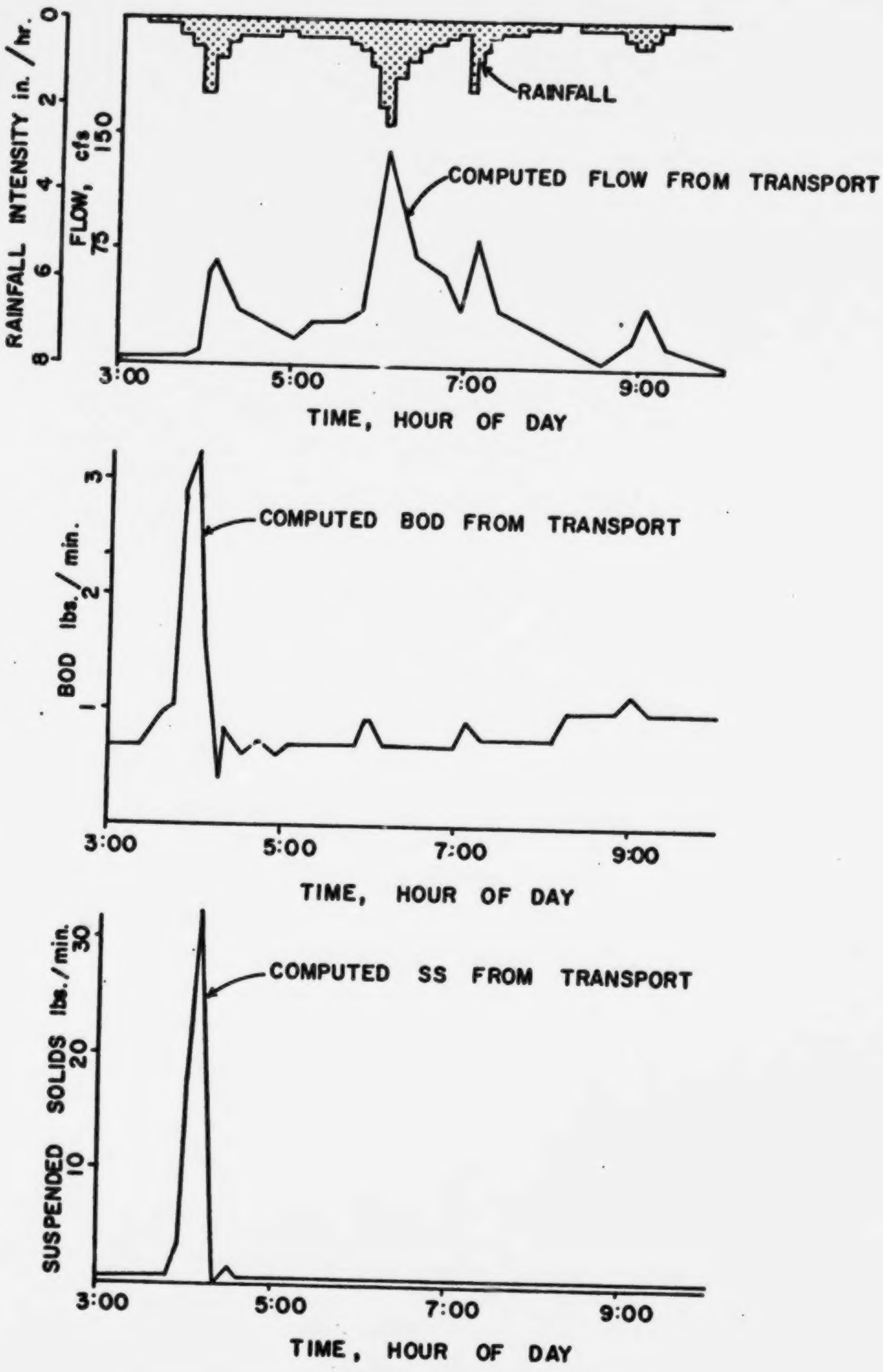


Figure 2-6
Runoff-Transport Simulation for Stevens Avenue.
Study 1, Storm 3.

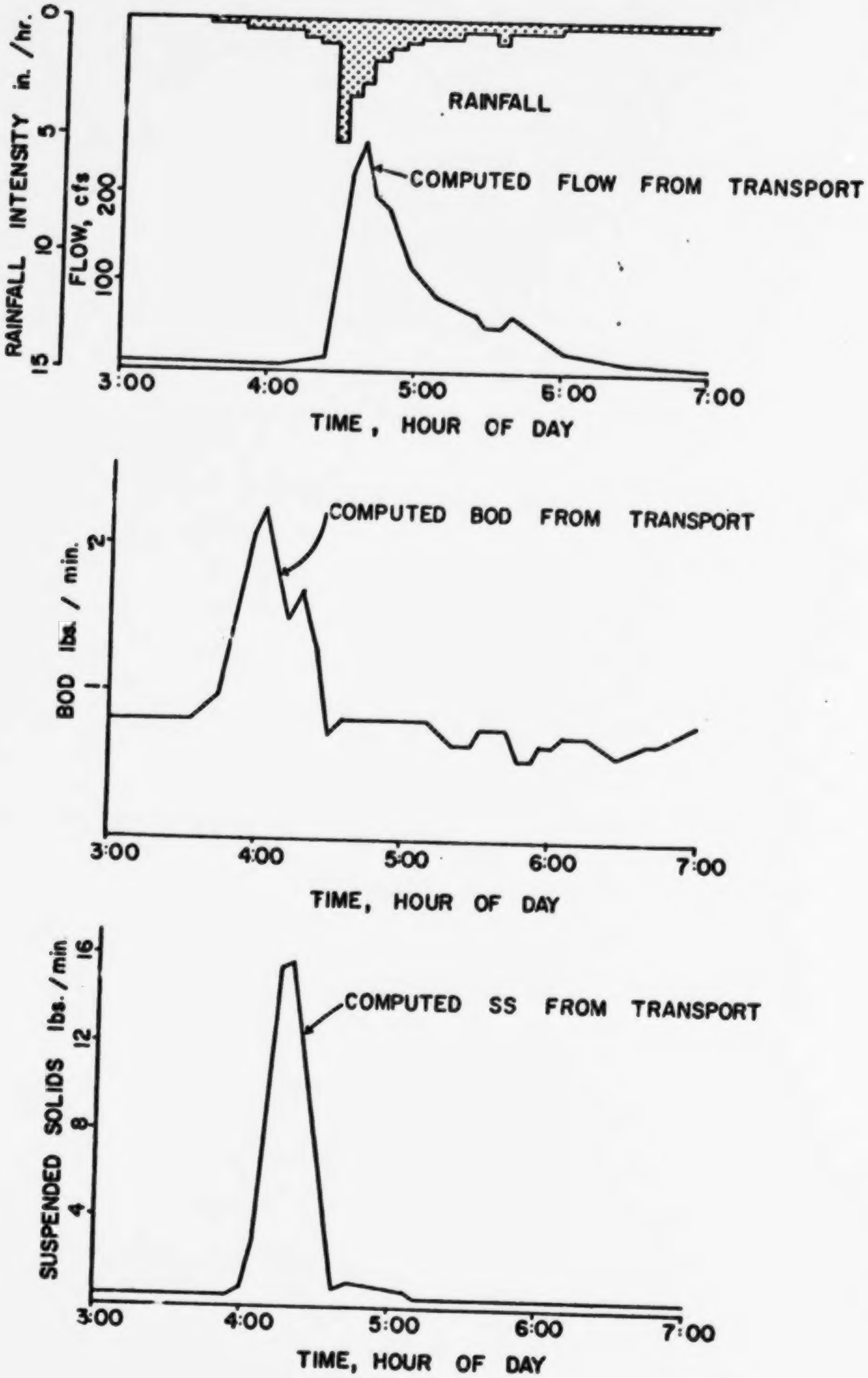


Figure 2-7
Runoff-Transport Simulation for Stevens Avenue.
Study 1, Storm 4.

results of subsequent studies indicate that actual overflows are generally predicted adequately by computer runs. Quality predictions are more variable.

Results of this study were used by APWA in sizing the swirl concentrator. A design flow of this device was established at 150 cfs.

Computer simulation studies were also conducted for all six storms to evaluate the effect of the swirl concentrator-underground silo facilities on the combined overflow quality. The results of Storm Nos. 5 and 6 are shown on Figures 2-8 and 2-9 respectively. As illustrated in these figures, the quality of the overflow is significantly improved through the installation of the swirl concentrator-underground silo.

Study No. 2.--This study consisted of a storm that began in the morning of August 27, 1971 and continued almost 30 hours to the morning of the next day. It resulted in varying amounts of rainfall throughout the city averaging more than 3.5 inches. The results of the computer simulation were similar to those obtained from Study No. 1, and for this reason are not included herein. Again, no measurements were taken during this study.

Study No. 3.--This study is based on a relatively minor rainfall event of March 22, 1972. This study is of special importance, however, because it is one of the types most frequently experienced in terms of intensity of rainfall. It is also one for which relatively complete verification data such as rainfall, flow readings and samples were collected. The rainfall is shown in Figure 2-10 along with results of the computer simulation showing overflow quantity and quality.

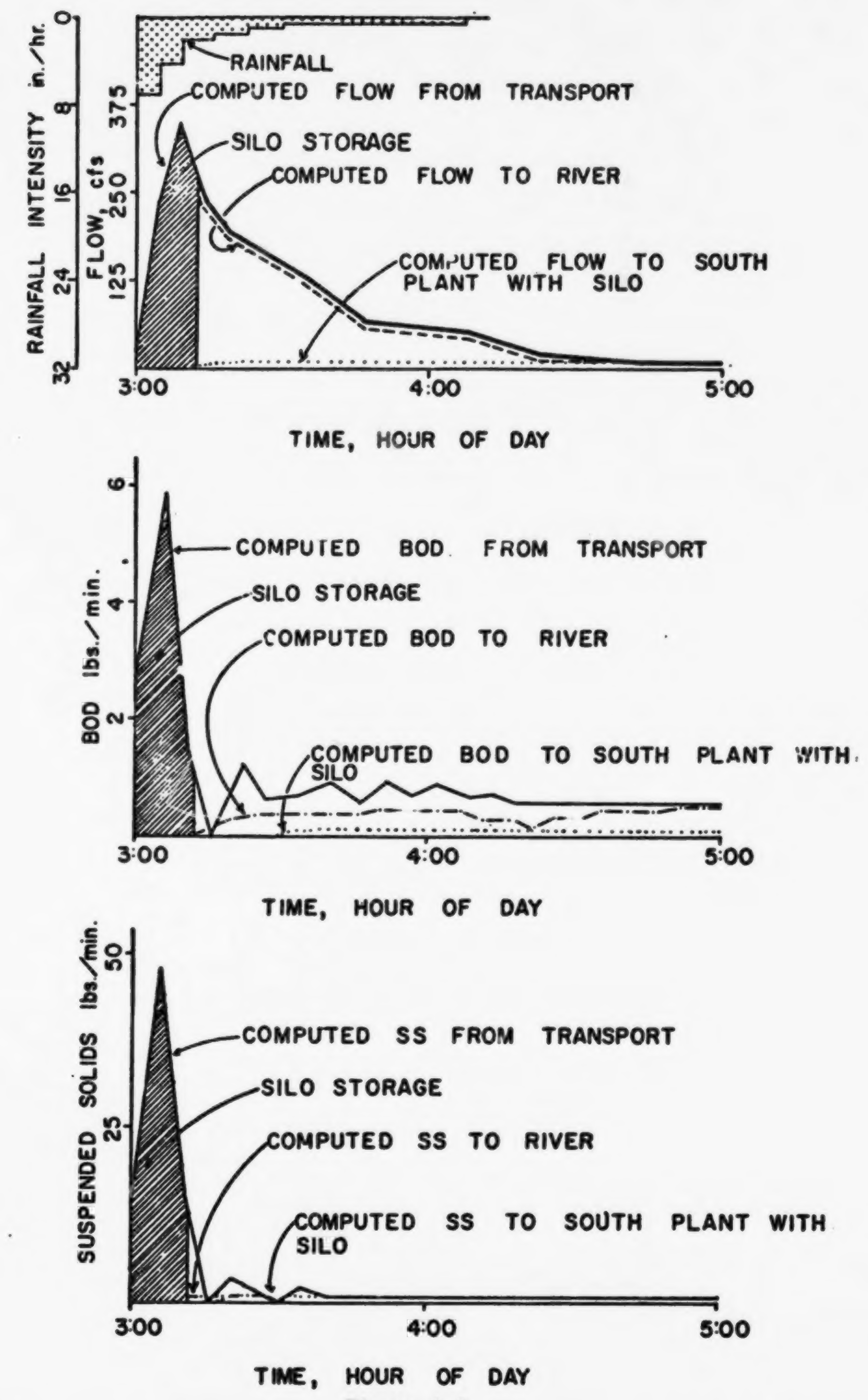


Figure 2-8
Runoff-Transport Simulation for Stevens Avenue with Silo and Swirl Concentrator.
Study 1, Storm 5.

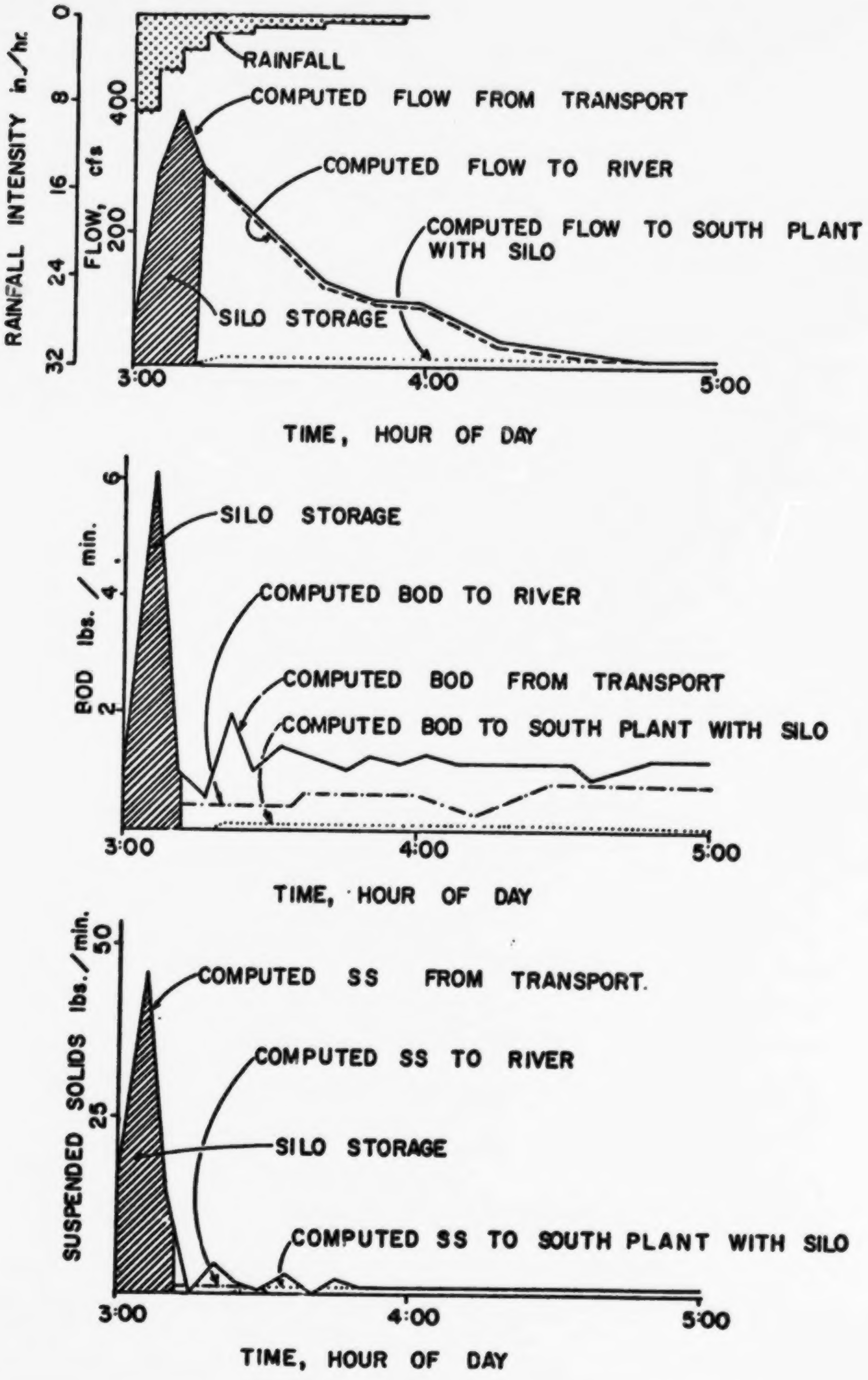


Figure 2-9
Runoff-Transport Simulation for Stevens Avenue with Silo and Swirl Concentrator.
Study 1, Storm 6.

Shown in the same illustration are the actual quantity and quality measurements of the overflow. It can be seen that agreement between the computer simulation and the actual measurements of flow is fairly good considering the degree of accuracy of the input data as well as that of the measurements. The agreement between the computed and measured quality parameters is not as good as for flows.

Computer simulations were also conducted on this study to determine the effect of the swirl concentrator-underground silo system. These results are also shown in Figure 2-10. With the silo system, the Model indicates no overflow in the Conestoga Creek.

Study No. 4.--This study is based on a storm that occured on November 29, 1971. This study is also of importance from the standpoint of Model verification as overflow measurements were conducted during this storm. The rainfall and results of the computer simulation for this storm are presented in Figure 2-11 along with the actual measurements for comparison. Again, it can be seen that agreement between the actual measurements and predicted results is fairly good. The predicted results of the swirl concentrator-underground silo system are also shown in Figure 2-11.

3. RUNS IN THE NORTH AND SOUTH DISTRICT

Limited computer simulations were also conducted for the North and South drainage districts. The North district was subdivided into 66 catchments and the South district into 104 catchments. The sewer layouts for the North and South districts are shown in Figures 2-12 and 2-13.

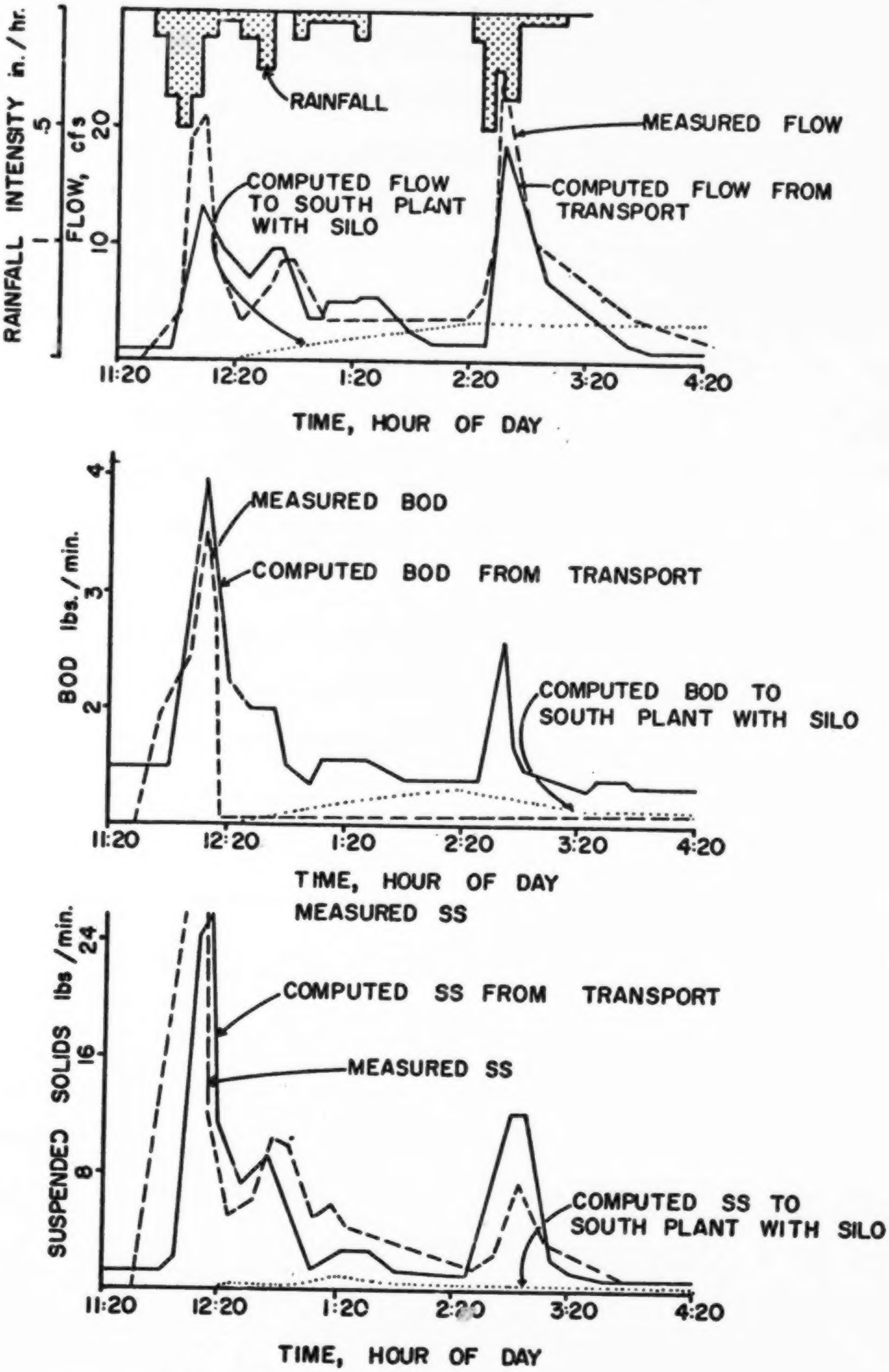


Figure 2-10
Runoff-Transport Simulation for Stevens Avenue with Silo and Swirl Concentrator. Study 3. No Overflow to River Since Silo Capacity not Exceeded.

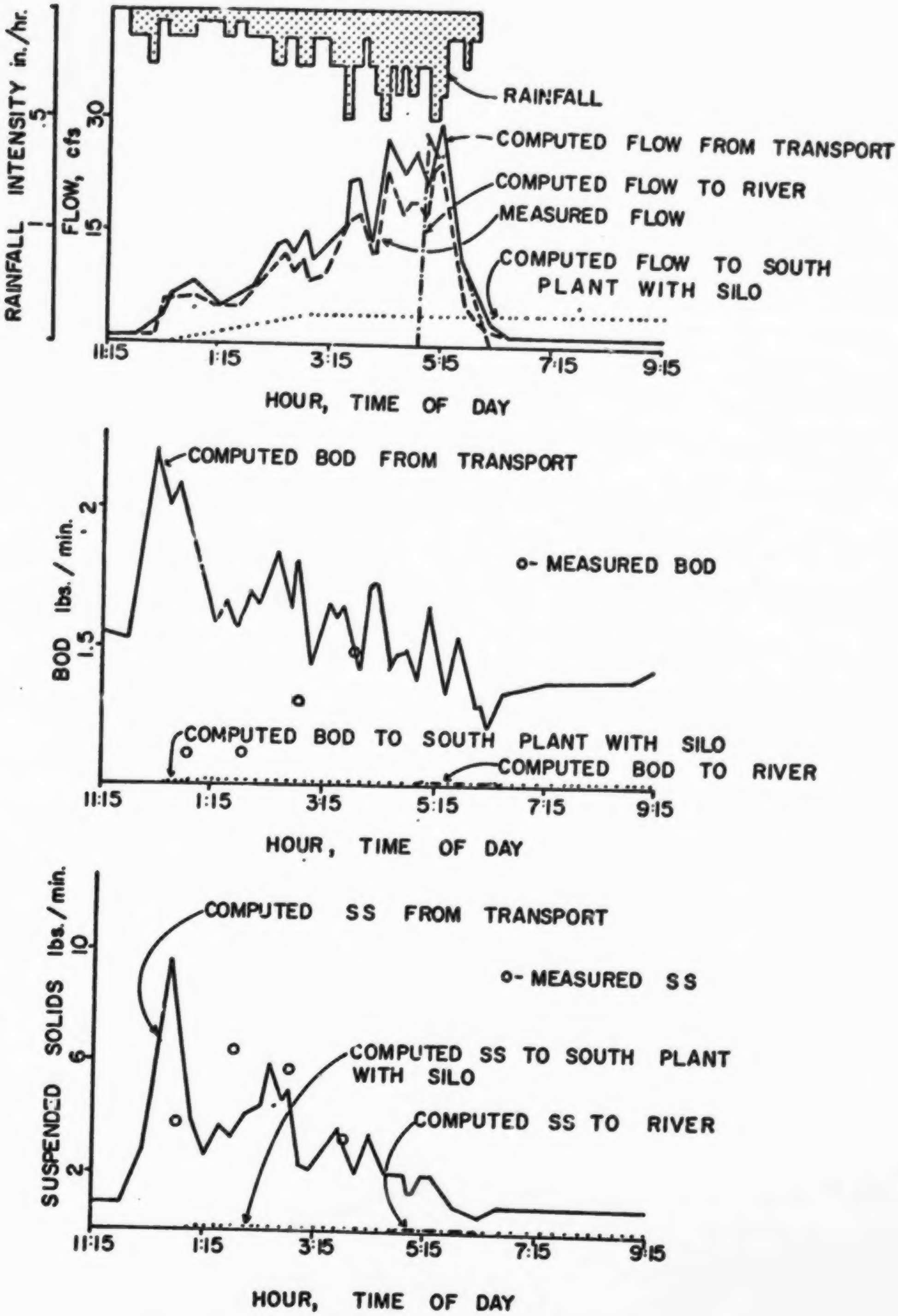


Figure 2-11
Runoff-Transport Simulation for Stevens Avenue with Silo and Swirl Concentrator.
Study 4.

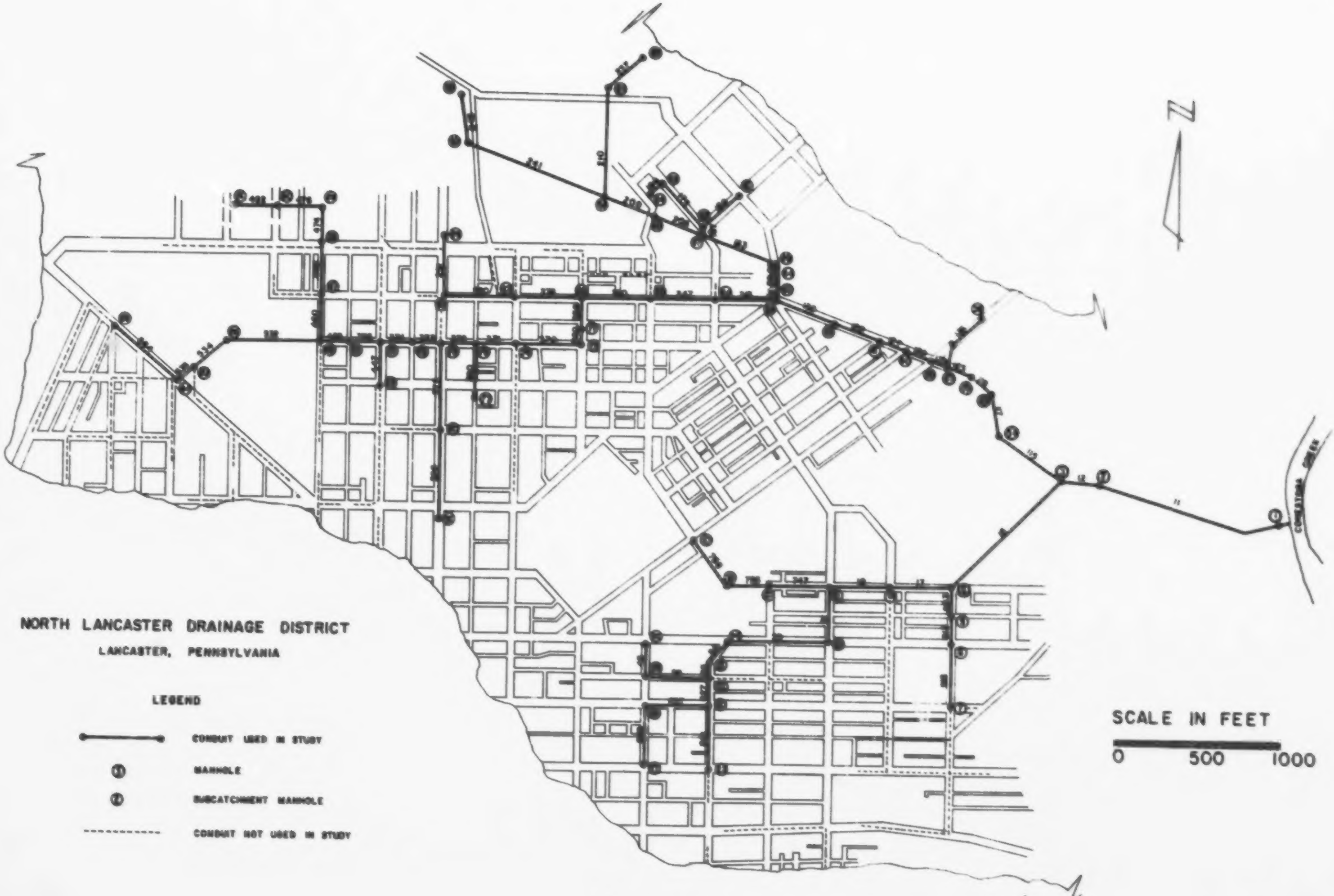

N
CONESTOGA CREEK
NORTH LANCASTER DRAINAGE DISTRICT
LANCASTER, PENNSYLVANIA
LEGEND
CONDUIT USED IN STUDY
MANHOLE
SUBCATCHMENT MANHOLE
CONDUIT NOT USED IN STUDY
SCALE IN FEET
0
500
1000

Figure 2-12
North Drainage District with Runoff-Transport Numbering System.

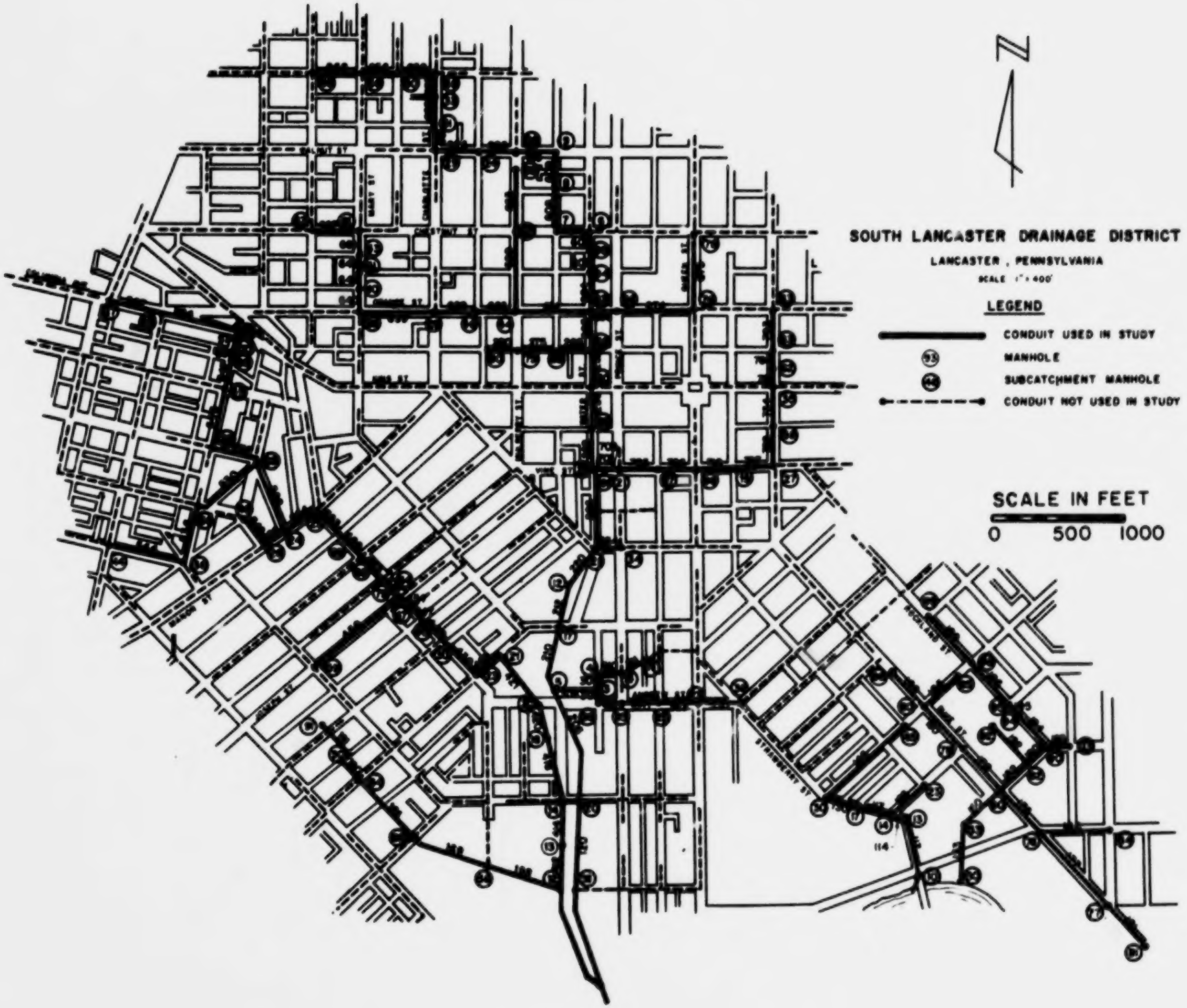

N
SOUTH LANCASTER DRAINAGE DISTRICT
LANCASTER, PENNSYLVANIA
SCALE 1" = 400'
LEGEND
CONDUIT USED IN STUDY
MANHOLE
SUBCATCHMENT MANHOLE
CONDUIT NOT USED IN STUDY
SCALE IN FEET
0
500
1000

Figure 2-13
South Drainage District with Runoff-Transport Numbering System.

Results of computer simulation for Study No. 3 for the North district are presented in Figure 2-14 and for the South district in Figure 2-15. The North district outfall is located at point 1 while the South district outfall is located at point 12 as shown in Figure 2-11.

An examination of these figures shows that for a rainfall event equivalent to Study No. 3, overflow from the North district would be about 100 cfs and from the South district, 160 cfs. The BOD and SS discharged to the river would be 7,075 pounds and 9,696 pounds from the North District and 4,468 pounds and 10,006 pounds respectively for the South district.

4. EFFECT ON RECEIVING WATER

To simulate the effect of the overflow on the Conestoga Creek, Receiving Water Model was run on the entire city for the Study No. 3. The manner in which various districts were combined is shown on Figure 2-16. In conducting this run, the swirl concentrator was used at Stevens Avenue while Refined Storage and Treatment Model, as described elsewhere, was utilized to simulate the existing biological treatment at the North and South plants. The silo was deleted in order to have an overflow at Stevens Avenue outfall since the installation of the silo prevents any overflow for rainfall event equivalent to Study No. 3.

The reaeration coefficient for the Conestoga Creek was computed from a formula by O'Connor and Dobbins (1958). Results of the Receiving Water Model are shown in Figures 2-17 through 2-20. Figure 2-17 shows

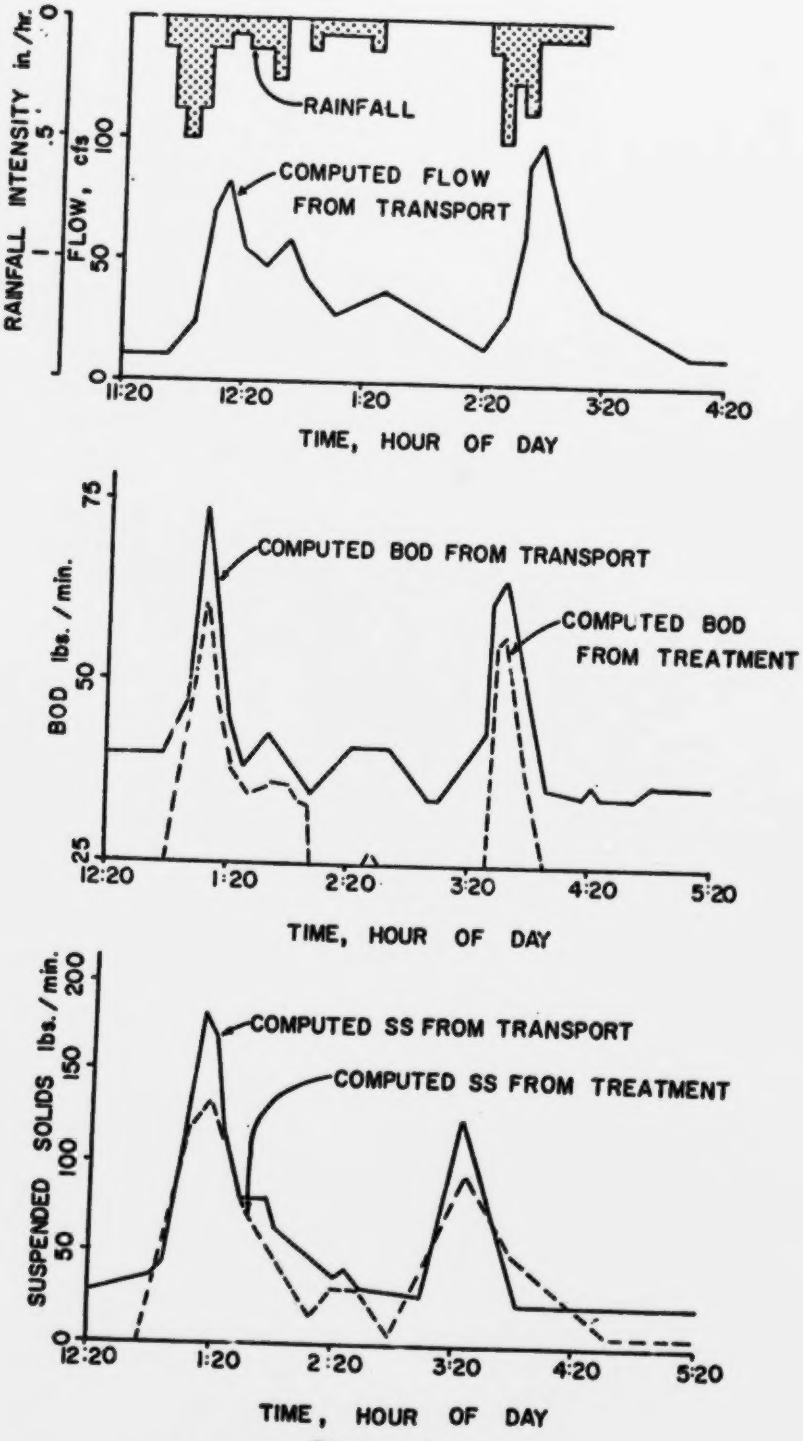


Figure 2-14
Simulation of North Drainage District.
Study 3.

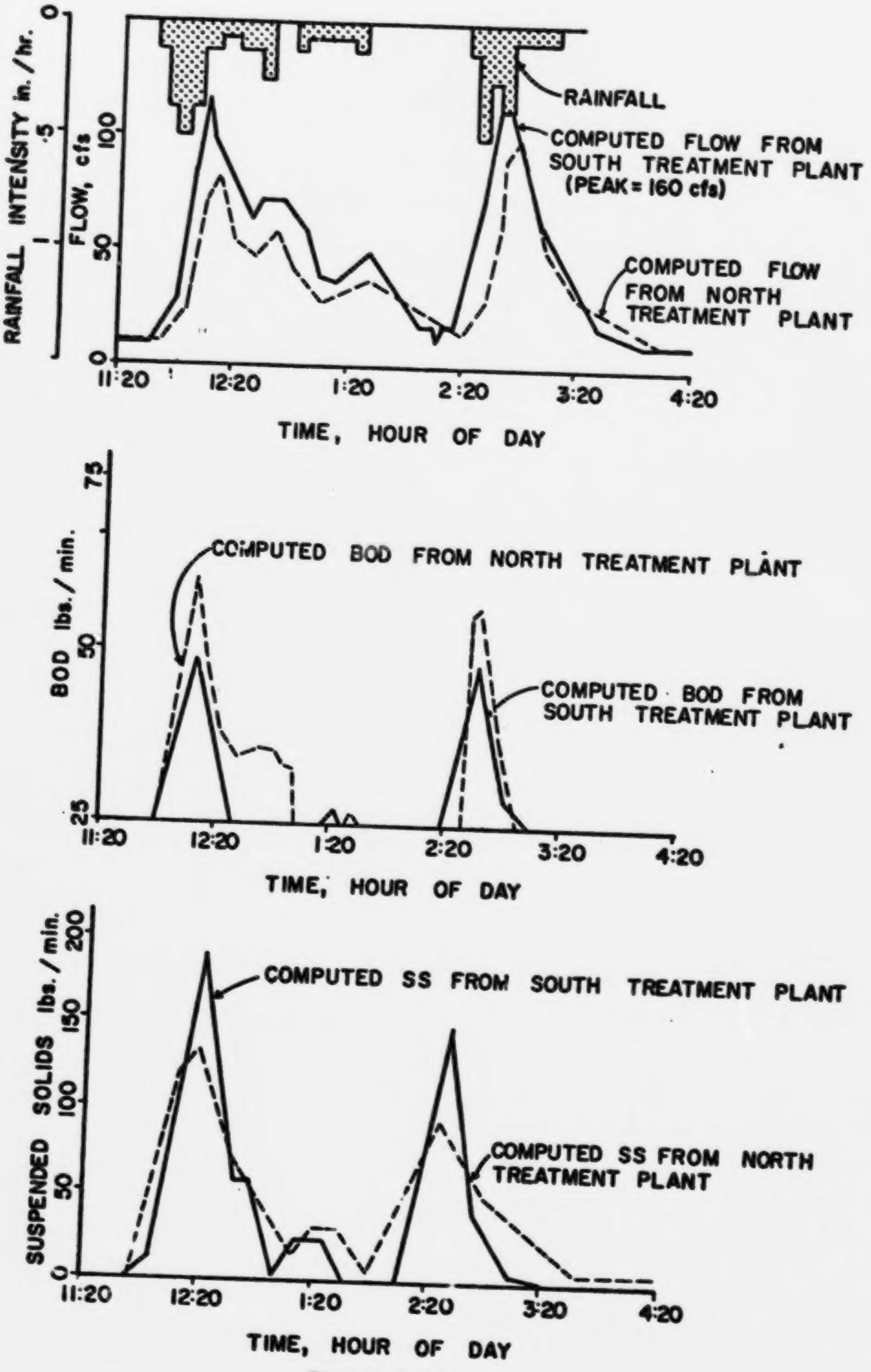


Figure 2-15
Simulation of North and South Drainage Districts.
Study 3.

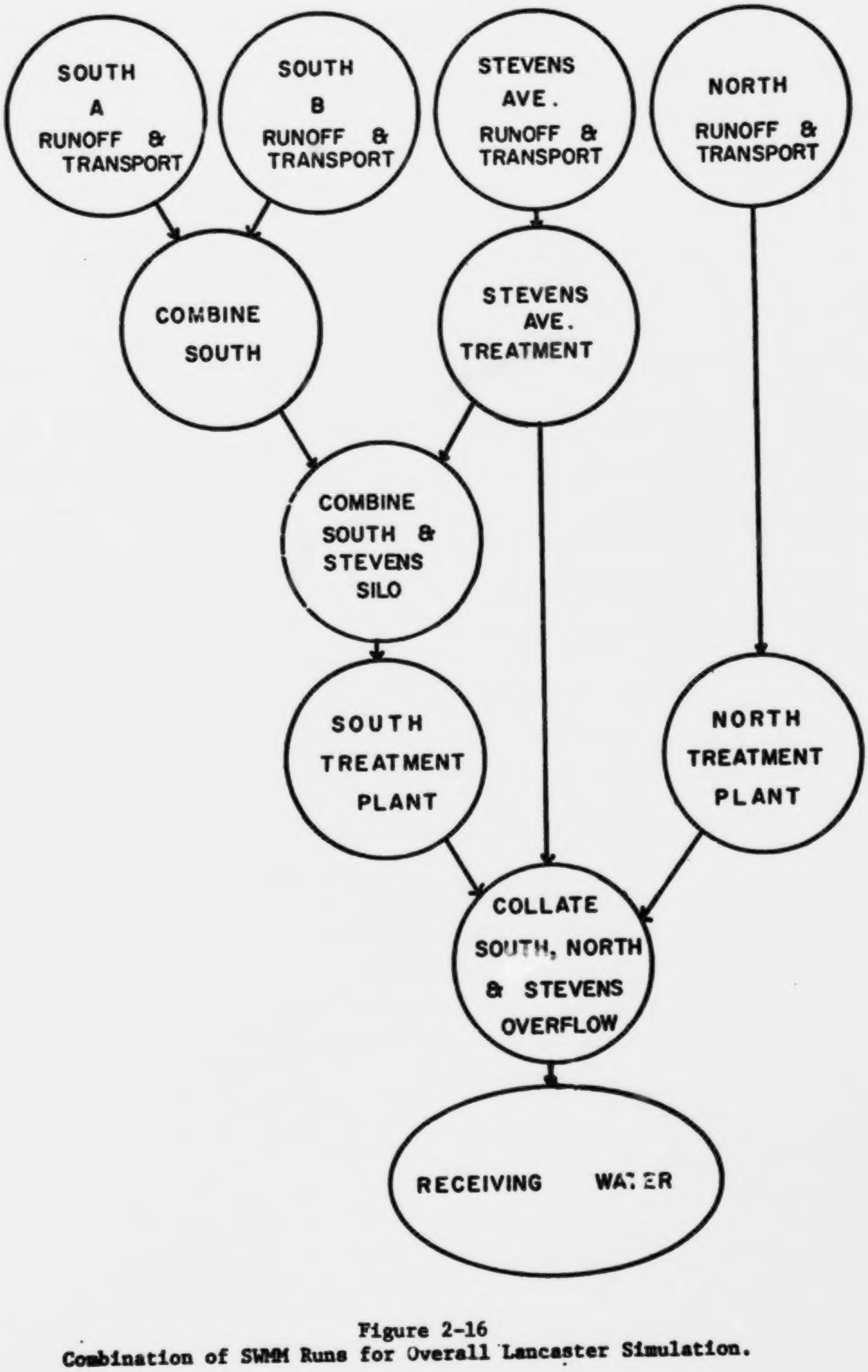

SOUTH
A
RUNOFF &
TRANSPORT
SOUTH
B
RUNOFF &
TRANSPORT
STEVENS
AVE.
RUNOFF &
TRANSPORT
NORTH
RUNOFF &
TRANSPORT
COMBINE
SOUTH
STEVENS
AVE.
TREATMENT
COMBINE
SOUTH &
STEVENS
SILO
SOUTH
TREATMENT
PLANT
NORTH
TREATMENT
PLANT
COLLATE
SOUTH, NORTH
& STEVENS
OVERFLOW
RECEIVING WATER

Figure 2-16
Combination of SWMM Runs for Overall Lancaster Simulation.

DO profiles 24 and 48 hours after the storm inception, while Figure 4-28 shows the BOD profile for the same period. Suspended solids and coliform are shown in Figures 2-19 and 2-20 respectively. Initial values used to simulate the Receiving Water Model are listed in Table 2-1:

TABLE 2-1
PARAMETERS USED FOR SIMULATING RECEIVING WATER MODEL

Dissolved Oxygen in Conestoga Creek (all junctions)	10.0 mg/1
BOD in Conestoga Creek (all junctions)	5.0 mg/1
Suspendid Solids in Conestoga Creek (all junctions)	10.0 mg/1
Coliform in Conestoga Creek (all junctions)	50/100 ml
Decay Coefficient (BOD)	0.20/day
Reaeration Coefficient	1.50/day
Flow in Conestoga Creek (entering junction 1)	700 cfs

5. SUMMARY

The above discussion can be summarized as follows:

1) The SWMM was able to predict fairly accurately the quantity as well as quality of the combined overflow for the Stevens Avenue district in Lancaster.

2) The installation of the swirl concentrator and the silo will result in substantial improvement in the quality of the overflow at Stevens Avenue, provided the full-scale performance of the swirl concentrator is comparible to the results obtained in laboratory studies by APWA.

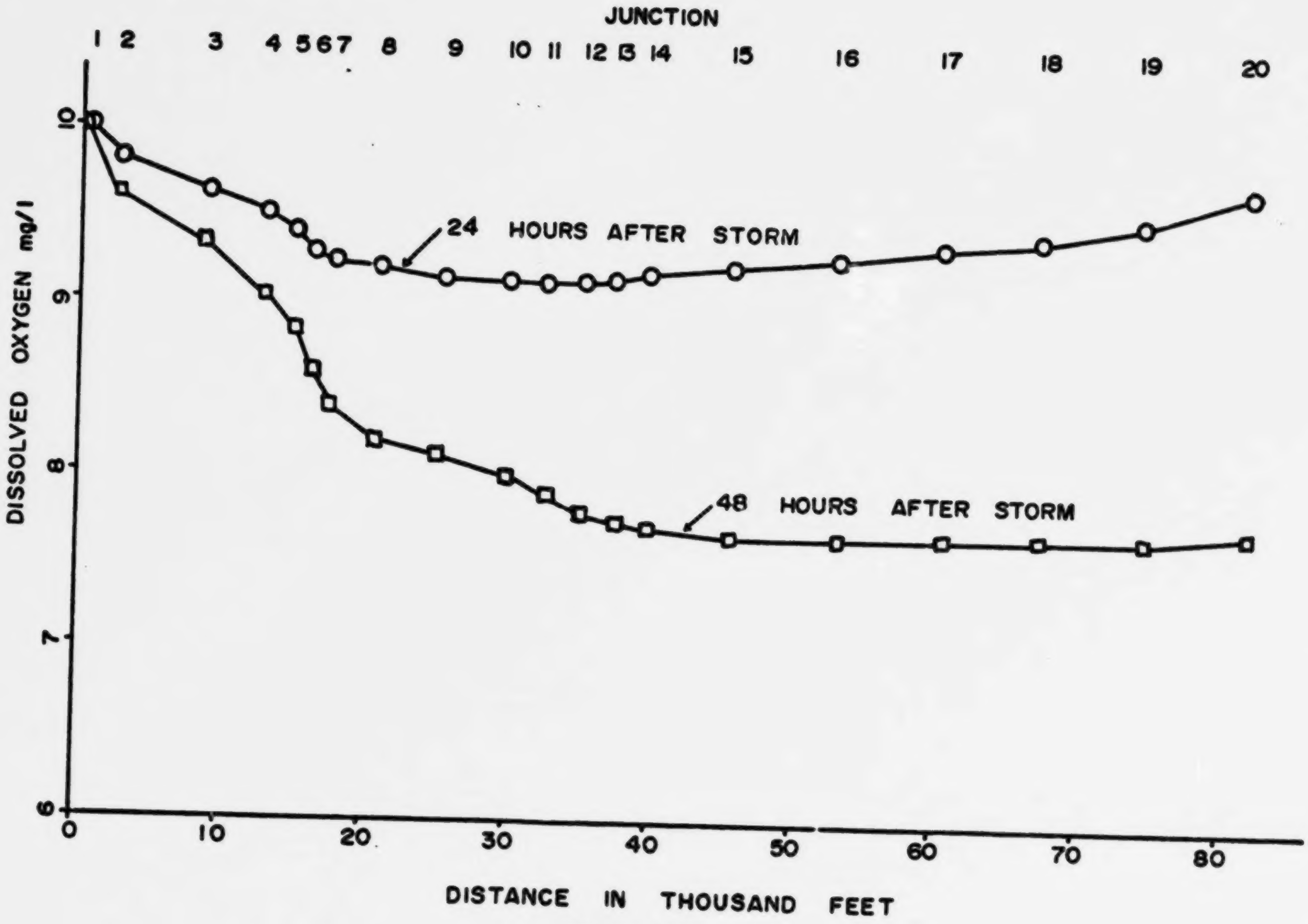


Figure 2-17
Dissolved Oxygen Profiles Along the Conestoga Creek.
Study 3.

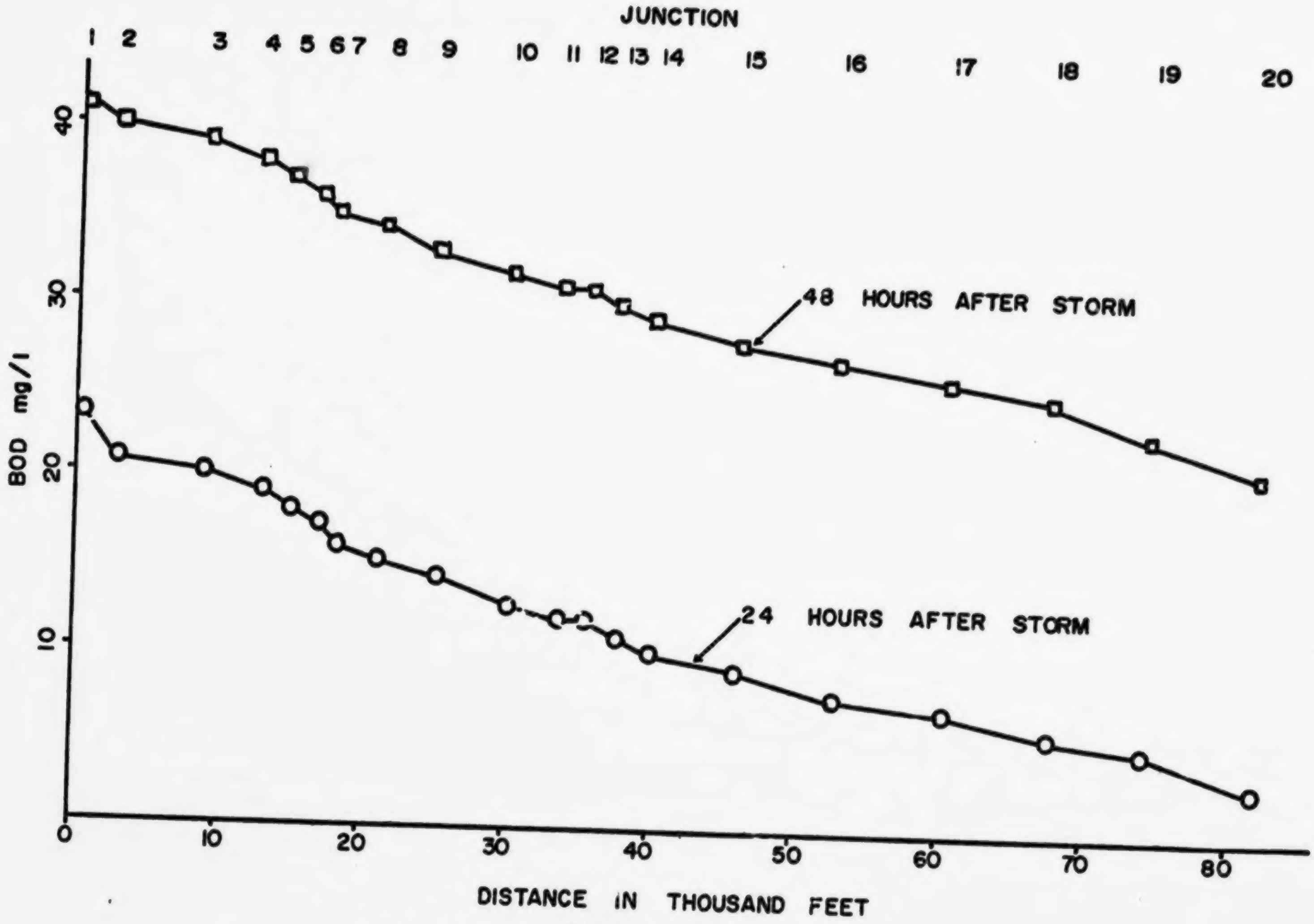


Figure 2-18
BOD Profiles Along the Conestoga Creek.
Study 3.

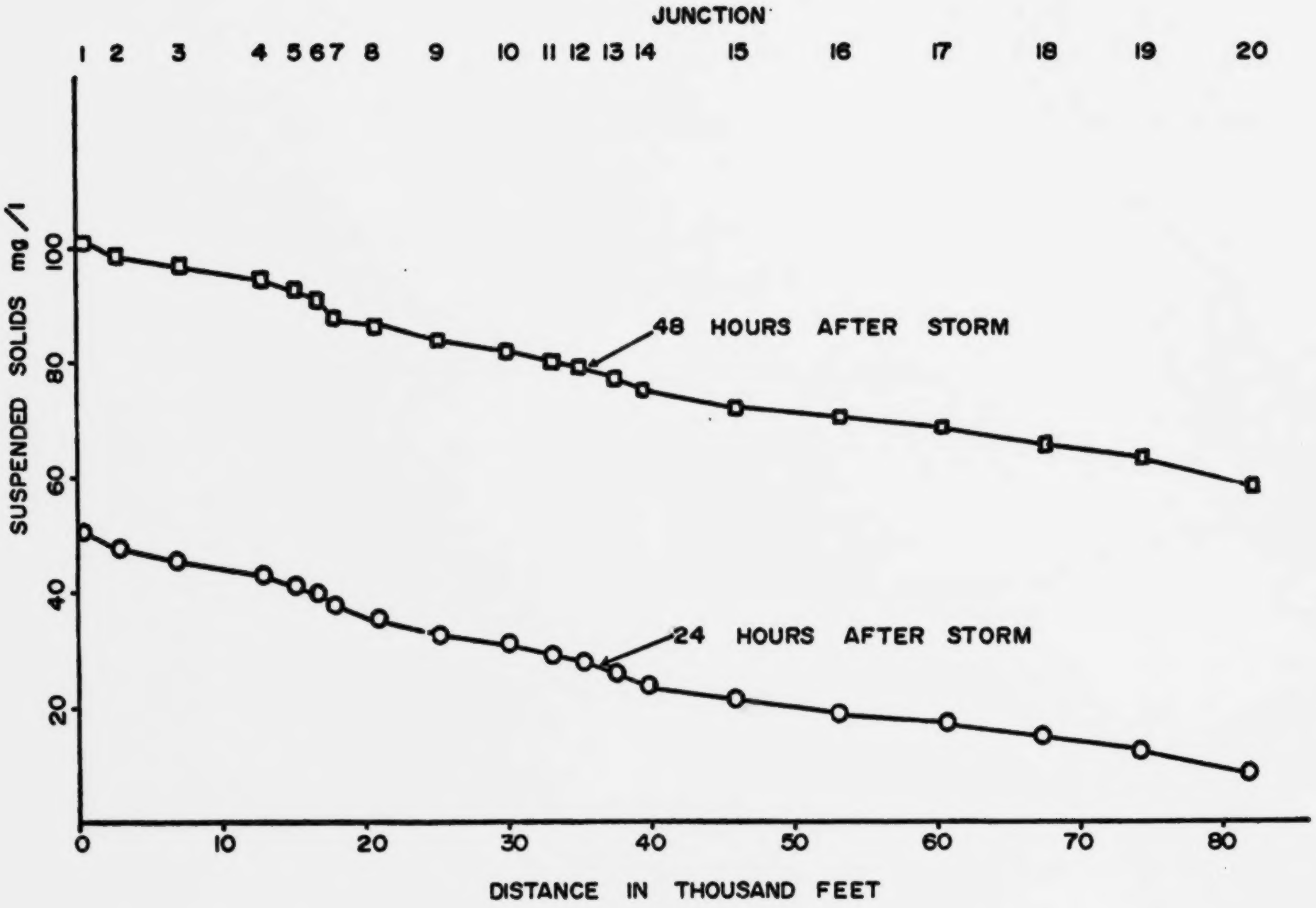


Figure 2-19
Suspended Solids Profiles Along the Conestoga Creek.
Study 3.

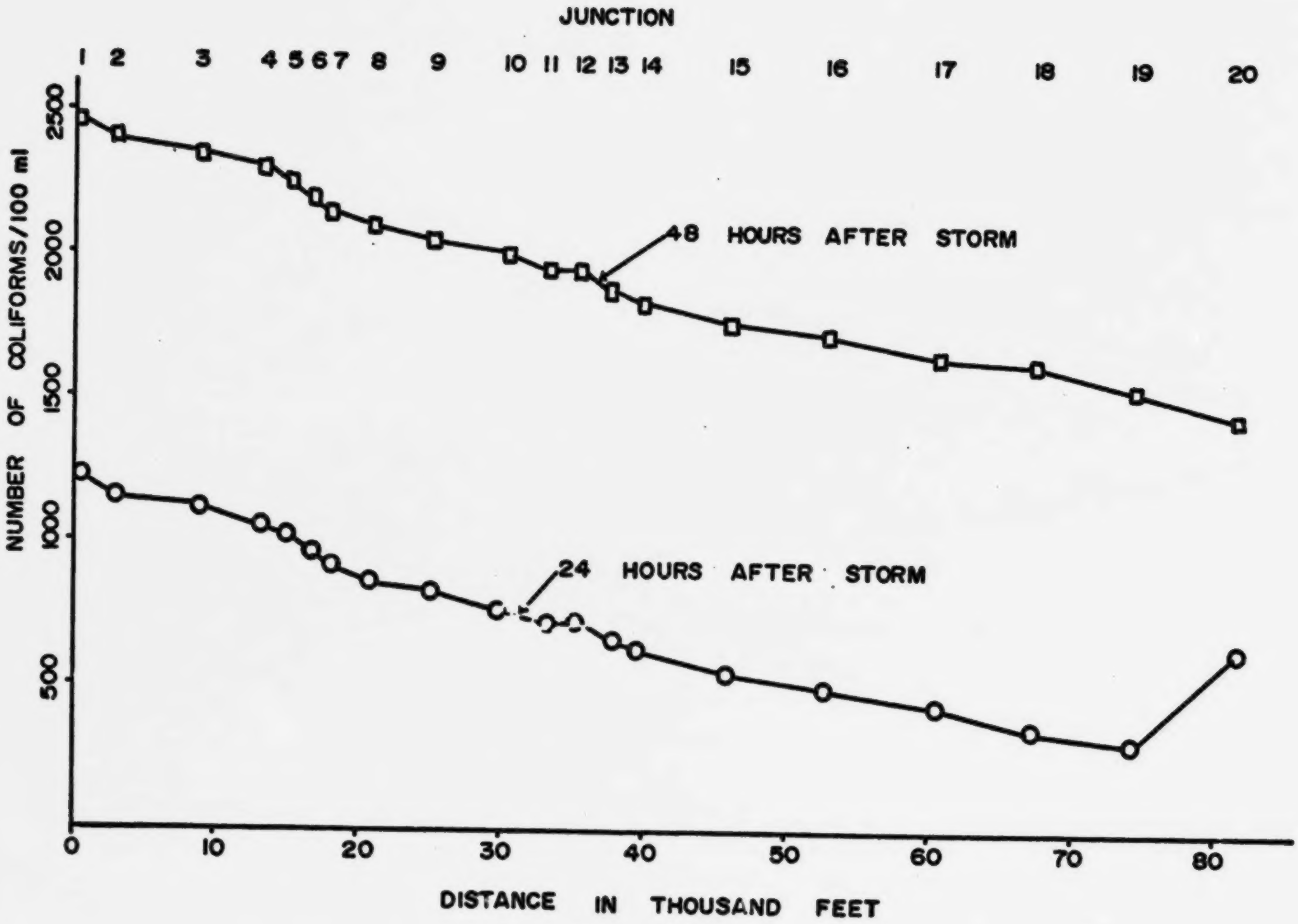


Figure 2-20
Coliform Profiles Along the Conestoga Creek.
Study 3.

3. REFERENCES

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b. "Volume II, Verification and Testing," Rept. No. 11024DOC08/71
c. "Volume III, User's Manual," Rept. No. 11024DOC09/71
d. "Volume IV, Program Listing," Rept. No. 11024DOC10/71

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4. ACKNOWLEDGEMENTS

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